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JOURNAL
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News Letter

ELEVENTH HOUR SCRAMBLES

TO FACILITATE the consideration of some of the papers and reports scheduled for the program of the 27th annual convention, the Publications Committee found itself virtually committed to a publication program which became more and more "impossible" with the approach of the final press date for the February JOURNAL.

Such reports for instance as those which appear in this and the February JOURNAL as a result of the labors of Committee 105, Reinforced Concrete Column Investigation, (a total of 162 pages) are unwieldy for adequate convention presentation unless, with report already in print, the presentor may assume in his audience some measure of conversance with the subject matter and thus feel free to confine himself to major significances.

The valuable and bulky contributions which in general characterize the contents of these two latest issues of *Proceedings* pages, and the late availability of a considerable amount of the total matter involved, combined to make almost imperative some unusual publication measures.

With the cooperation of printers and authors in prompt proof revision, the February JOURNAL with nearly double an average monthly quota of pages, was in the mails five days ahead of the scheduled publication date and this the March issue becomes available as an aid to convention discussion nearly three weeks early.

The eleventh hour scramble by authors, editors, printers, and proofreaders, gives occasion for emphasis of the Institute's new situation: We are no longer wholly dependent on an *annual* forum; we have added ten more. We now have ten objectives a year—ten JOURNAL issues, each of them with an audience at least four times as great as our largest annual convention.
—EDITOR.

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Second Progress Report on Column Tests at the University of Illinois

BY F. E. RICHART* AND G. C. STAEHLE†

SERIES 3—COLUMNS UNDER LONG TIME LOADS

1. *Introduction.*—This report is a continuation of the progress report given in the A. C. I. JOURNAL, February, 1931[1], on the results of the Reinforced Concrete Column Investigation. The previous report described the part of the investigation being conducted at the University of Illinois in cooperation with the American Concrete Institute and Lehigh University, gave details of the methods used in making and testing columns, and included a preliminary report on the tests of Series 1, "A Study of End Conditions of Column," and Series 2, "A Study of Method of Loading."[1] The outline of the entire investigation, prepared by Committee 105 of the Institute is given in the 1930 report of Committee 105[2].

The tests at the University of Illinois have been carried on as a cooperative research project of the Engineering Experiment Station, under the administrative direction of Dean M. S. Ketchum, Director of the Station, and Professor M. L. Enger, Head of the Department of Theoretical and Applied Mechanics. Acknowledgment is made of the assistance given by L. R. Tucker and T. G. Taylor, members of the Experiment Station Staff, in the testing work and the reduction of the data.

Due to the lack of time for consideration of the test results by the other members of Committee 105, this report (like the previous one) has been prepared as an individual report by those

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†Associate Engineer, Structural and Technical Bureau, Portland Cement Association, Chicago.

[1]Progress Report on Column Tests at the University of Illinois, *Proc. A. C. I.*, V. 27, p. 731, 1931.

[2]Progress Report of Committee 105, *Proc. A. C. I.*, V. 26, p. 601, 1930.

in charge of the tests at the University. However, it is desired to give full acknowledgment of the help of all members of the Committee, who participated in planning the tests, furnished many facilities for the work and advised concerning all phases of the investigation.

2. *Object and Outline of Tests of Series 3.*—It has been realized for many years that the shrinkage of concrete will set up stresses in a reinforced concrete column, and great emphasis was placed on this effect of shrinkage and the similar one of time yield or plastic flow by McMillan[3] in 1921. The purpose of the present series of tests was (1) to make a systematic study of the amount of shrinkage and plastic flow produced in reinforced concrete columns under the ordinary design load, continuously applied, and (2) having induced a redistribution of stresses in a column due to the action of shrinkage and flow over a period of a year, to note the effect upon the strength of the column as found by testing it to failure under the usual method of loading.

The series as outlined included 108 columns, of which 60 are to be held for a year under design load and 48 are duplicate columns held in similar storage but under no load. The design loads have been computed from the column formulas of the A. C. I. Joint Building Code, except for four columns, which have been loaded according to the design formula of the 1916 building code for New York City.

A list of the columns of Series 3, including the design loads used, is given in Table 1. All columns were made and cured in the moist room for 56 days. They were then placed in one of the following four conditions, of loading and storage, to be maintained for a period of one year:

(1) Under design load, in air of laboratory.
(2) Under no load, in air of laboratory.
(3) Under design load, in moist room.
(4) Under no load, in moist room.

The columns stored in air are subject to temperatures of 70 to 90 deg. F., and to relative humidities of 40 to 90 per cent. Those in the moist room are at temperatures of 70 deg. ± 4 deg., in a saturated atmosphere. Systematic strain measurements are

[3]A Study of Column Test Data, by F. R. McMillan, Proc. A. C. I., V. 17, p. 150-181, 1921

TABLE 1. DATA OF COLUMNS OF SERIES 3

Two companion columns of each type, except plain columns. All columns have 8-in. core diameter, 8¼ in. overall diameter, and are 5 ft. long. All columns cured 56 days in moist room, preceding the one-year period of loading and strain measurement. Design loads based on A. C. I. Joint Building code formulas, except for 4 columns indicated by *, which were loaded according to 1916 New York Code formula. All reinforcement is of intermediate grade steel. Columns marked † were loaded in 1931, all others in 1930.

Design Concrete Strength Lb. per sq. in.	Percentage of Reinforcement		Design Load on Column Lb.	Columns Loaded			Columns not Loaded	
	Vertical	Spiral		No.	Cylinder Strength Lb. per sq. in.	Date	No.	Cylinder Strength Lb. per sq. in.
			Air Storage					
2000	1.57	2.00	38,000	2	2220	8–21	2	2115
	3.98	0	37,200	2	2375	6–11	2	2535
	3.98	1.24	64,000	2	2245	7–31	2	2280
	3.98	2.00	64,000	2	2520	7– 7	2	2740
	5.94	2.00	90,000	2	2175	8– 7	2	2210
3500	1.57	0	47,200	2	3850	5–28	..	....
	1.57	1.24	52,400	2	3405	8–11	..	....
	1.57	2.00	52,400	2	3475	8–12	2	3770
	1.57	2.00	67,000*	2	3555	8–14	..	....
	3.98	0	55,200	2	4110	6– 4	2	4280
	3.98	1.24	79,000	2	3700	7–17	2	3795
	3.98	2.00	79,000	2	4240	6–25	2	3965
	5.94	0	81,200	2	3975	6–19	..	....
	5.94	1.24	108,600	2	3620	8– 4	..	....
	5.94	2.00	108,600	2	3520	8–15	2	4125
	5.94	2.00	75,200*	2	3545	8–26	..	....
5000	1.57	2.00	59,400	2	5570	8–19	2	5515
	3.98	0	72,000	2	5230	6– 9	2	5600
	3.98	1.24	96,000	2	5125	7–23	2	5215
	3.98	2.00	96,000	2	6105	7– 2	2	5860
	5.94	2.00	130,400	2	5690	8– 5	2	5435
2000	0	0	26,700	2	....	2–12†	2	....
3500	0	0	46,600	2	....	1–29†	2	....
5000	0	0	66,800	2	....	3– 2†	2	....
2000	0	0	0	..	....	Sept.	6	2420
3500	0	0	0	..	....	Sept.	7	4015
5000	0	0	0	..	....	Sept.	6	5185
			Moist Storage					
2000	3.98	0	37,200	2	2640	6–17	2	2485
	3.98	1.24	64,000	2	2285	7–29	2	2090
	3.98	2.00	64,000	2	2360	7–10	2	2475
3500	3.98	0	55,200	2	3605	5–29	2	3810
	3.98	1.24	79,000	2	3400	7–15	2	3490
	3.98	2.00	79,000	2	3965	6–23	2	3945
5000	3.98	0	72,000	2	4685	6– 5	2	4515
	3.98	1.24	96,000	2	5310	7–21	2	5320
	3.98	2.00	96,000	2	5435	6–30	2	5390
2000	0	0	0	..	....	Sept.	4	2110
3500	0	0	0	..	....	Sept.	6	3590
5000	0	0	0	..	....	Sept.	4	5610

being made on all of the columns throughout the year using a 10-in. Whittemore strain gage.

Table 1 shows that three grades of concrete were used. These were designed to produce 56-day cylinder strengths of 2000, 3500, and 5000 lb. per sq. in. Three percentages of longitudinal reinforcement, roughly 1.5, 4 and 6, and two percentages of spiral, 1 and 2, were employed in the various columns.

In addition to the 108 reinforced columns originally scheduled 45 plain columns have been made and six of these will be held under constant load for a year. These plain columns were added to provide information on the flow and shrinkage of unreinforced concrete, and to serve as control specimens with regard to the strength of the concrete in the reinforced columns at the end of a year. Four 6 by 12-in. control cylinders were also made with each column, two to be tested at the age of 56 days, and two to be tested a year later when the column is tested to failure.

3. *Materials.*—Details regarding the properties of the concrete and reinforcing steel and a description of the making of columns are given in the progress report published in the A. C. I. JOURNAL, February, 1931.

Average properties of the three concrete mixtures employed are as follows:

Designed strength, lb. per sq. in. (56-day moist curing)	2000	3500	5000
Proportions of mix, by weight	1:4.05:4.95	1:3.27:3.98	1:2.25:2.75
Water-cement ratio, by volume	1.40	1.10	0.80
Average slump, in.	2.9	4.3	5.1

The actual 56-day cylinder strengths are given in Table 1. The materials used in all columns were Lehigh portland cement, Lincoln, Ill. sand and Attica, Ind. gravel. The concrete was mixed 4 minutes in a one-bag batch mixer.

Both the longitudinal and spiral reinforcement were of intermediate grade steel. The selection of bars to provide the three percentages of longitudinal steel was as follows:

1.57 per cent	4 ½-in. round	Yield Point, 45,600 lb. per sq. in.
3.98 per cent	8 ½-in. square	Yield Point, 53,400 lb. per sq. in.
5.94 per cent	4 ⅝-in. round 4 ¾-in. round	Yield Point, 39,300 lb. per sq. in. 51,100 lb. per sq. in.

The reinforcing spirals, of 8-in. outside diameter, were made of hot-rolled rod. The 1.24 per cent spirals were of No. 5 rod, 1.35 in. pitch, and the 2.0 per cent spirals were of ¼-in. rod, 1.19 in. pitch. The stress at which a unit elongation of the spiral rod of 0.5 per cent was produced (in tension tests of coupons) has been taken as a "useful limit" for the material; the average values found were 49,400 lb. per sq. in. for the No. 5 rod and 48,200 lb. per sq. in. for the ¼-in. rod. The respective ultimate tensile strengths were 79,500 and 74,200 lb. per sq. in.

4. *Making and Curing Test Columns.*—Preparatory to making columns, the reinforcing bars were milled to exact length and wired to the spirals or ties to form a unit. The columns were cast in tight steel forms. A tamping rod and a light air hammer were used in placing the concrete. Precautions were taken to bring both ends of the reinforcing bars into the planes of the concrete bearing faces of the columns.

Forms were removed 24 hours after a column was cast. Due to the fact that the forms were water-tight and the top of the column had been covered by a metal capping plate, the column surfaces were damp when the forms were removed and there had evidently been little or no loss of moisture from the column. Shortly after the removal of forms steel plugs were set to form gage lines on which to measure concrete deformations, and gage lines were laid out on the longitudinal steel. The column was then placed in the moist room or stored under wet burlap for one day, when all gage lines were drilled and initial strain measurements were taken. The exposed steel at gage holes was then given a coating of vaseline as a protection against corrosion and the columns were placed in the moist room to complete the 56-day period of moist curing. Strain measurements taken at the end of this period furnished a record of the volume change, usually a slight expansion, produced by about 54 days of moist storage.

5. *Method of Applying Continuous Loads.*—The apparatus for applying and holding a constant load on the columns may be described by reference to Fig. 1. Companion columns, tested in pairs to save apparatus, are confined between bearing plates by four steel tie rods. The desired tension is maintained in these

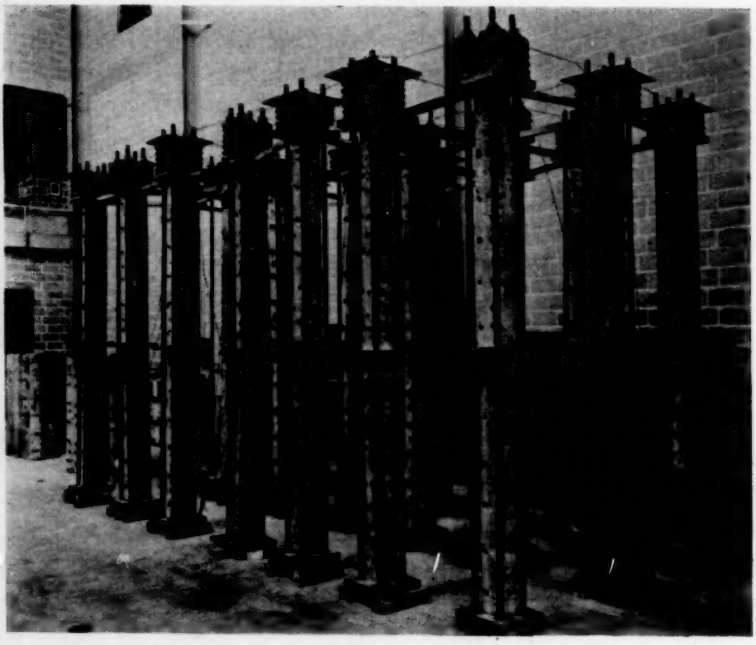

FIG. 1—VIEW OF COLUMNS OF SERIES 3 IN MATERIALS TESTING LABORATORY

rods by four heavy coiled compression springs, to be seen at the top of each rig in the figure. These springs were calibrated, after having been subjected to ten repetitions of their capacity load (for the purpose of relieving localized stresses and thus minimizing the permanent set which might occur under the long continued loading). The desired load was applied to the columns by tightening the nuts on the four tie-rods until the necessary shortening of each spring was reached. This shortening varied from 1.0 to 1.3 in. with the five sizes of springs used, and since errors in measurement on the two gage lengths on each of the four springs would normally be compensating, the design loads are believed to be accurate to within 1 per cent. A complete set of strain measurements was taken on the columns immediately before and after applying the load, and at regular intervals thereafter.

Measurements on gage lines on the springs were made coincident with the reading of strains in the columns. With shrinkage and yielding of the columns a slight decrease in the indicated spring load resulted. One complete adjustment of all loading rigs was made after about three months of loading and since that time there has been little change in the applied load. At the time of the adjustment, several springs were removed and checked, but practically no permanent set of the springs could be detected.

It was anticipated that unequal strains might be produced on opposite sides of a column by eccentricity of load or by variations in the material of the column. To minimize eccentric loading and to stabilize the loading rig, the tie rods were run through holes in a bearing plate between columns at mid-height of the rig. There has been no appreciable evidence of bending or unequal yielding.

The tie-rods and plates shown in Fig. 1 were furnished for the work through the courtesy of the American System of Reinforcing and the Illinois Steel Co., Chicago.

6. *Working Loads.*—As previously noted, the columns were loaded at safe working stresses computed by use of the design formulas for tied and spirally reinforced columns of the A. C. I. Joint Building Code and for spirally reinforced columns of the 1916 New York City building code. This does not imply that the test columns met all of the design requirements of these two codes. For the purposes of the investigation it was considered desirable to use combinations of materials outside the restricted field of either code. Thus most of the columns have a percentage of spiral reinforcement in excess of one-fourth that of the longitudinal reinforcement, as required by the A. C. I. code, and one type of column has a proportion less than one-fourth. None of the columns have an appreciable protective shell of concrete outside the reinforcement. The New York code does not recognize a grade of concrete having a strength of 3500 lb. per sq. in., nor does it permit the use of more than 4 per cent of longitudinal reinforcement. Hence the use of these formulas in determining working loads is not in strict accordance with the apparent intent of the codes, but it is thought that the use that has been made of them has been reasonable and fairly conservative.

The working loads used with all columns are listed in Table 1. The loads for columns without spiral reinforcement were computed by use of the Joint Building Code formula for tied columns. For the plain concrete columns, no formula is applicable. The loads used were chosen to produce concrete stresses comparable with those existing in the reinforced columns in the early stages of loading. It is evident that further tests are needed to determine the flow of the different grades of concrete under various intensities of load.

7. *Flow and Shrinkage During a Period of Five Months.*—The end of the one-year loading period will be reached for the various reinforced columns during June, July and August, 1931. Similar periods for plain columns will extend to March, 1932. However, observations have been taken on all of the reinforced columns and some of the plain ones for five months or more, and it appears that the most rapid development of flow and shrinkage strains has taken place in this period. Figs. 2 to 9 show values of measured strains in the columns, beginning at the end of the 56-day curing period, when the columns were placed under load, and continuing for 5 months. Using a modulus of elasticity of 30,000,000 lb. per sq. in. these strains may be translated into stresses in the longitudinal steel. The curves have been grouped as much as possible according to the class of concrete used in the columns. Generally the time-deformation curves for each pair of companion columns is plotted on the same diagram with similar curves for the corresponding unloaded columns. The storage of columns in the air of the laboratory is designated as "Air Storage" while storage in the moist room is designated as "Moist Storage."

It may be noted that the observations plotted on these diagrams (excepting Fig. 9) represent the strains in reinforcing bars only. The strains on adjacent concrete gage lines have not yet been computed from the original readings, but it may be assumed that they will closely approximate those in the longitudinal steel.

A record was kept of temperatures and relative humidity in both the laboratory and moist rooms when all strain observations were taken. A recording thermometer in the moist room has

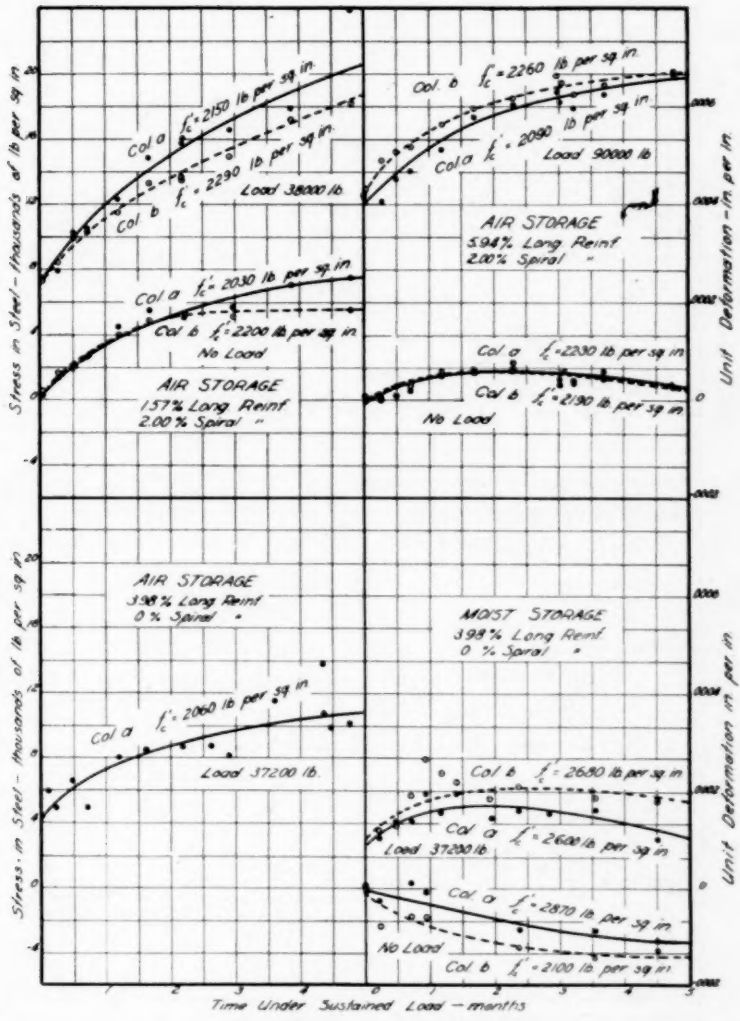
Col. b f'_c = 2260 lb per sq in.
Col a f'_c = 2090 lb per sq in.
Load 90000 lb
AIR STORAGE
5.94% Long Reinf
2.00% Spiral "
Col a f'_c = 2290 lb per sq in.
Col b f'_c = 2190 lb per sq in.
No Load
Col a f'_c = 2150 lb per sq in.
Col b f'_c = 2290 lb per sq in.
Load 38000 lb.
Col a f'_c = 2030 lb per sq in.
Col b f'_c = 2200 lb per sq in.
No Load
AIR STORAGE
1.57% Long Reinf
2.00% Spiral "
AIR STORAGE
3.98% Long Reinf
0 % Spiral "
Col a f'_c = 2060 lb per sq in.
Load 37200 lb.
MOIST STORAGE
3.98% Long Reinf
0 % Spiral "
Col b f'_c = 2680 lb per sq in.
Col a f'_c = 2620 lb per sq in.
Load 37200 lb
Col a f'_c = 2870 lb per sq in.
No Load
Col b f'_c = 2100 lb per sq in.
Stress in Steel - thousands of lb per sq in
Unit Deformation - in per in
Time Under Sustained Load — months

FIG. 2—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 2000-LB. CONCRETE

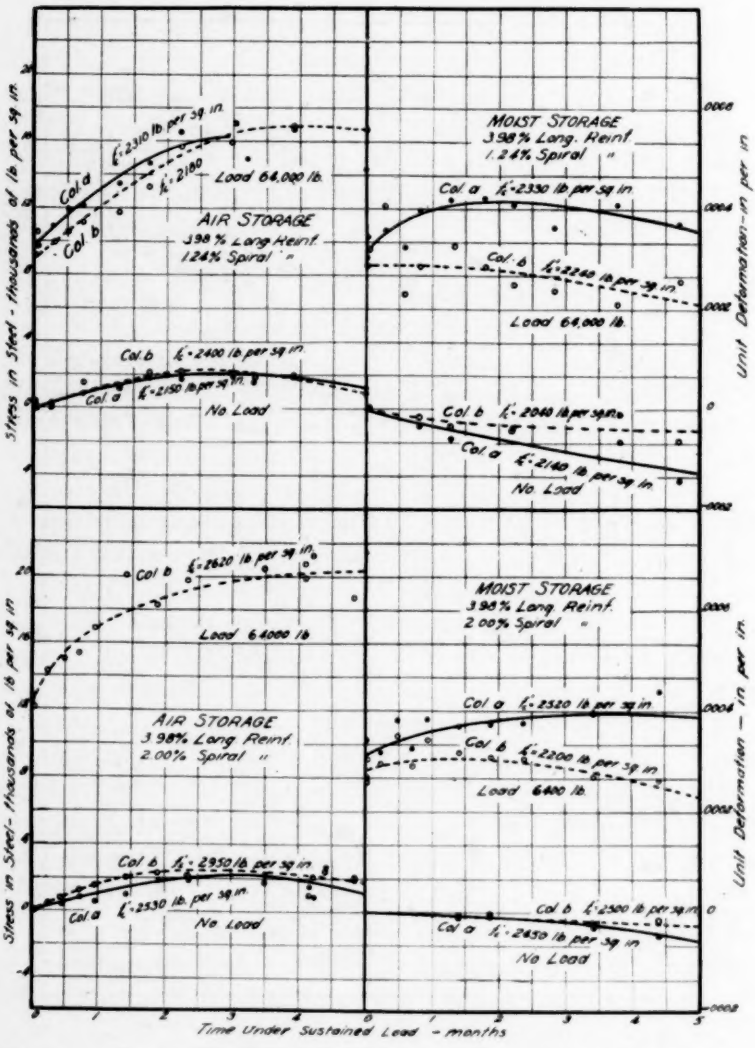

FIG. 3—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 2000-LB. CONCRETE

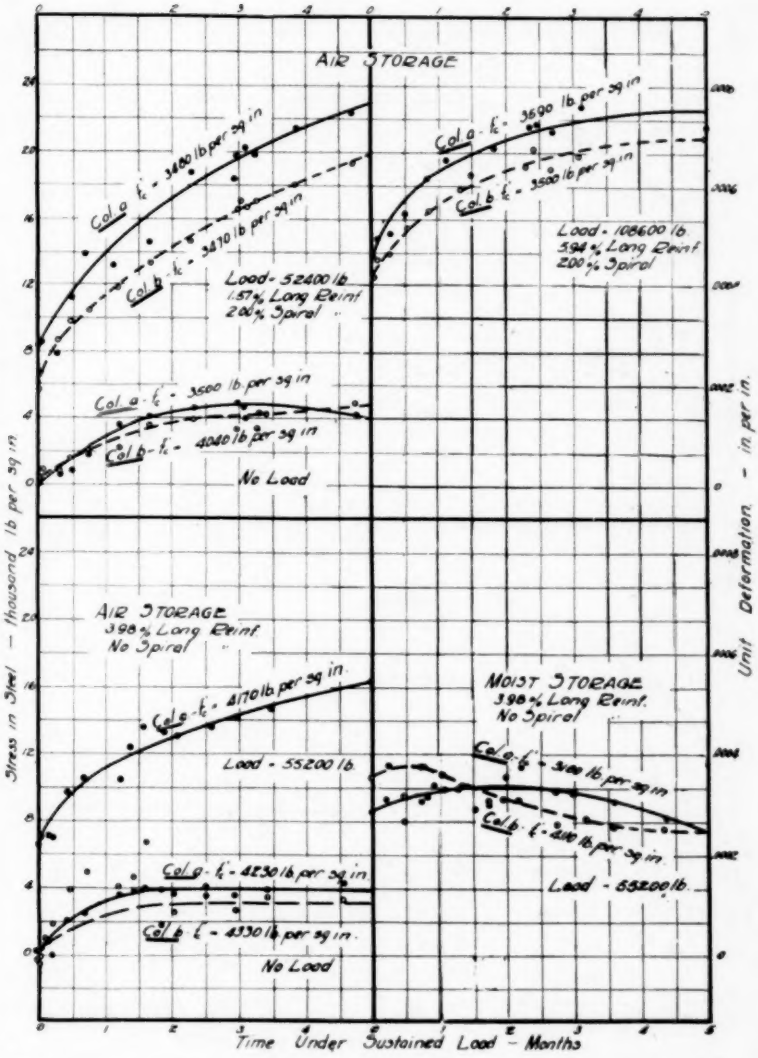
AIR STORAGE
Col a - f'c - 3480 lb per sq in
Col b - f'c - 3470 lb per sq in
Load - 52400 lb
1.57% Long Reinf
2.00% Spiral
Col a - f'c - 3600 lb per sq in
Col b - f'c - 4080 lb per sq in
No Load
Col a - f'c - 3690 lb per sq in
Col b - f'c - 3500 lb per sq in
Load - 108600 lb
3.98% Long Reinf
2.00% Spiral
AIR STORAGE
3.98% Long Reinf
No Spiral
Col a - f'c - 4110 lb per sq in
Load - 55200 lb
Col a - f'c - 4230 lb per sq in
Col b - f'c - 4330 lb per sq in
No Load
MOIST STORAGE
3.98% Long Reinf
No Spiral
Col a - f'c - 3400 lb per sq in
Col b - f'c - 2400 lb per sq in
Load - 65200 lb
Stress in Steel - thousand lb per sq in
Unit Deformation - in per in.
Time Under Sustained Load - Months

FIG. 4—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

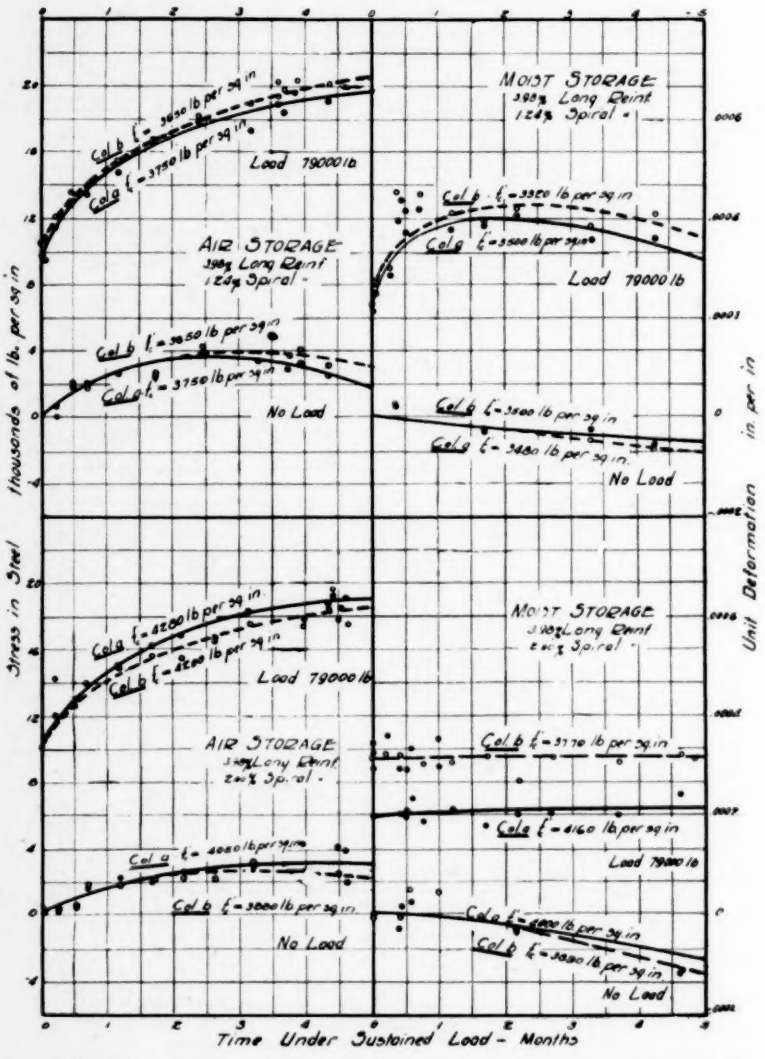
MOIST STORAGE
3.98% Long Reinf
1.24% Spiral
Col b f'c = 3650 lb per sq in
Col a f'c = 3750 lb per sq in
Load 79000 lb
AIR STORAGE
3.98% Long Reinf
1.24% Spiral
No Load
MOIST STORAGE
3.40% Long Reinf
2.00% Spiral
Col a f'c = 4200 lb per sq in
Col b f'c = 4200 lb per sq in
AIR STORAGE
3.40% Long Reinf
2.00% Spiral
Col a f'c = 4050 lb per sq in
Col b f'c = 3800 lb per sq in
Col b f'c = 3500 lb per sq in
Col a f'c = 3480 lb per sq in
Col b f'c = 3770 lb per sq in
Col a f'c = 4160 lb per sq in
Col a f'c = 4000 lb per sq in
Col b f'c = 3830 lb per sq in
Stress in Steel thousands of lb. per sq in
Unit Deformation in per in
Time Under Sustained Load - Months

FIG. 5—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

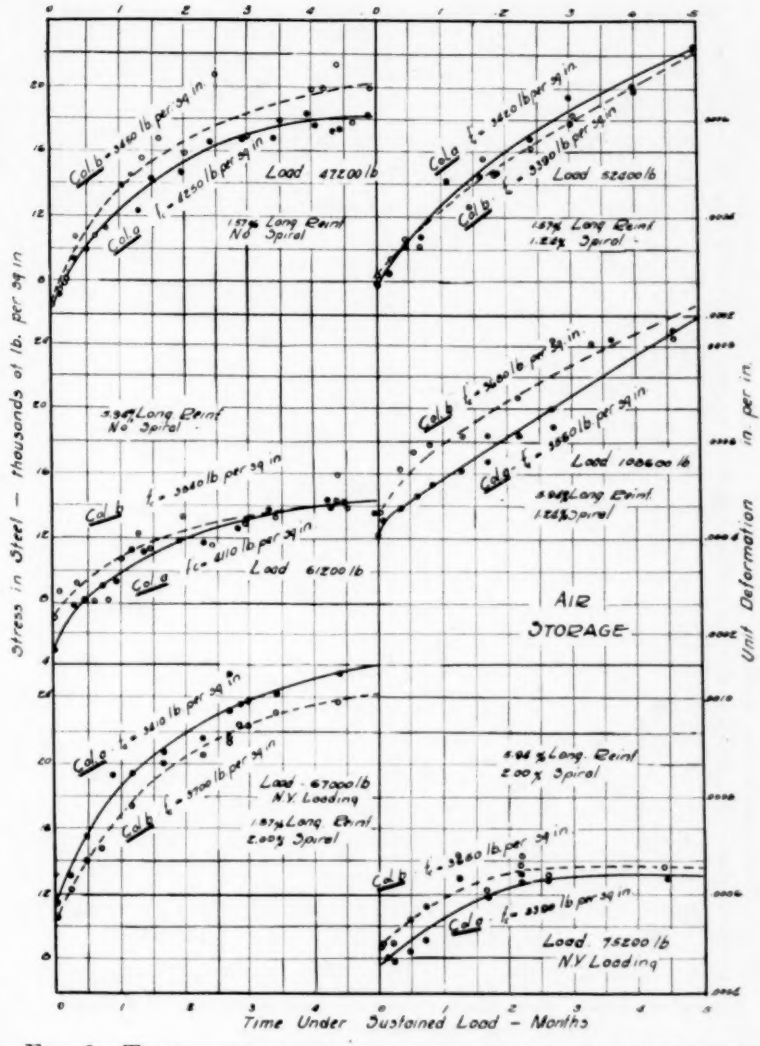

FIG. 6—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 3500-LB. CONCRETE

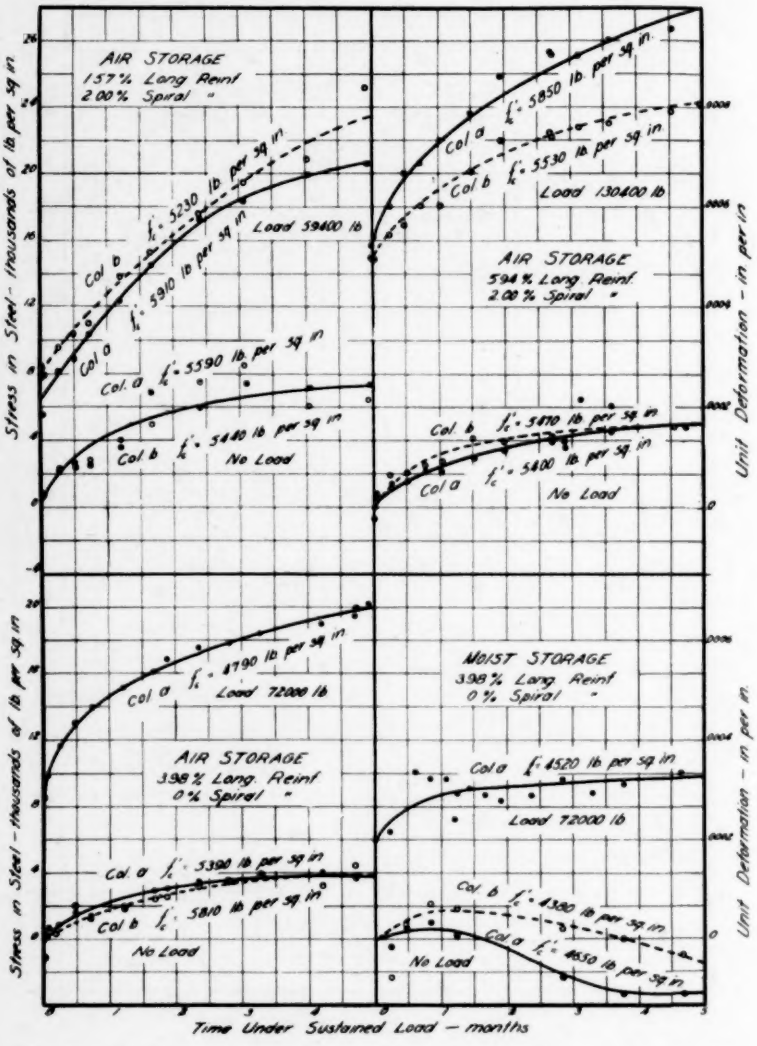
AIR STORAGE
1.57% Long Reinf
2.00% Spiral "
Col a f'c = 5230 lb per sq in
Load 59400 lb
Col b f'c = 5910 lb per sq in
Col a f'c = 5590 lb per sq in
Col b f'c = 5440 lb per sq in
No Load
AIR STORAGE
5.94% Long Reinf
2.00% Spiral "
Col a f'c = 5850 lb per sq in
Load 130400 lb
Col b f'c = 5530 lb per sq in
Col b f'c = 5470 lb per sq in
Col a f'c = 5400 lb per sq in
No Load
AIR STORAGE
3.98% Long. Reinf
0% Spiral "
Col a f'c = 4790 lb per sq in
Load 72000 lb
Col a f'c = 5390 lb per sq in
Col b f'c = 5810 lb per sq in
No Load
MOIST STORAGE
3.98% Long. Reinf
0% Spiral "
Col a f'c = 4520 lb per sq in
Load 72000 lb
Col b f'c = 4380 lb per sq in
Col a f'c = 4650 lb per sq in
No Load
Stress in Steel - thousands of lb per sq in
Unit Deformation - in per in
Time Under Sustained Load - months

FIG. 7—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 5000-LB. CONCRETE

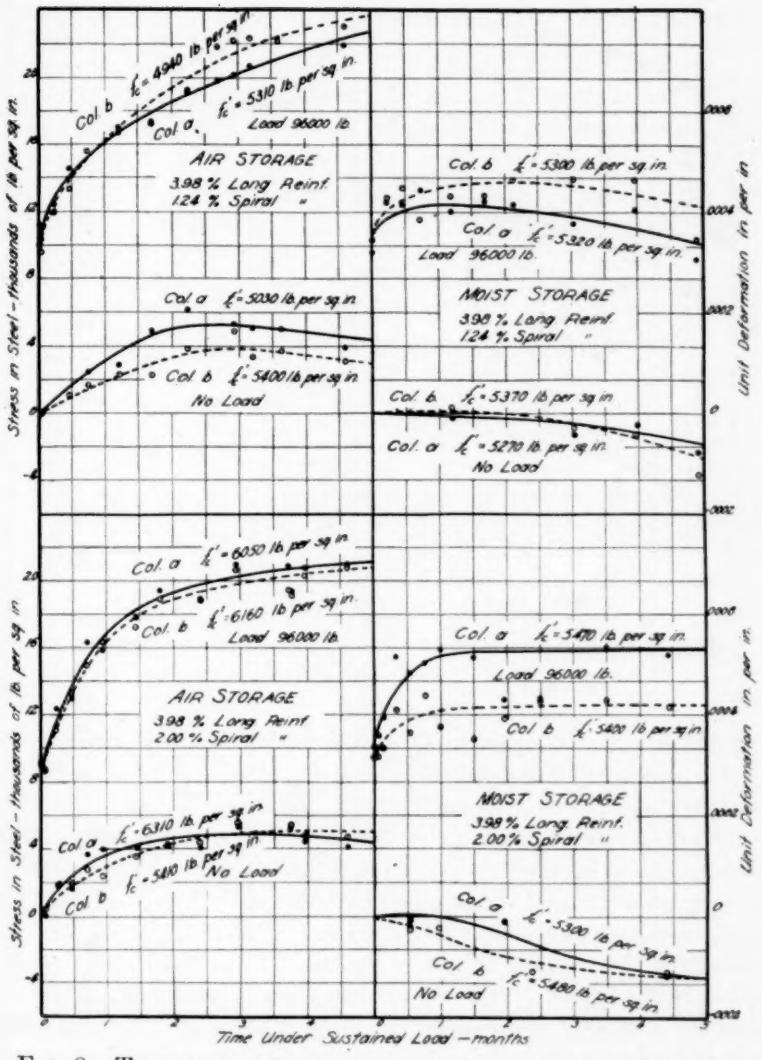
Col. b f'c = 4940 lb per sq in
Col. a f'c = 5310 lb per sq in.
Load 96000 lb.
AIR STORAGE
3.98 % Long. Reinf.
1.24 % Spiral "
Col. a f'c = 5030 lb per sq in.
Col. b f'c = 5400 lb per sq in.
No Load
Col. b f'c = 5300 lb per sq in.
Col. a f'c = 5320 lb per sq in.
Load 96000 lb.
MOIST STORAGE
3.98 % Long. Reinf.
1.24 % Spiral "
Col. b f'c = 5370 lb per sq in.
Col. a f'c = 5270 lb per sq in.
No Load
Col. a f'c = 6050 lb per sq in.
Col. b f'c = 6160 lb per sq in.
Load 96000 lb.
AIR STORAGE
3.98 % Long. Reinf.
2.00 % Spiral "
Col. a f'c = 6310 lb per sq in.
Col. b f'c = 5410 lb per sq in.
No Load
Col. a f'c = 5470 lb per sq in.
Load 96000 lb.
Col. b f'c = 5400 lb per sq in.
MOIST STORAGE
3.98 % Long. Reinf.
2.00 % Spiral "
Col. a f'c = 5300 lb per sq in.
Col. b f'c = 5480 lb per sq in.
No Load
Stress in Steel - thousands of lb per sq in.
Unit Deformation in per in.
Time Under Sustained Load - months

FIG. 8—TIME-DEFORMATION CURVES FOR COLUMNS MADE WITH 5000-LB. CONCRETE

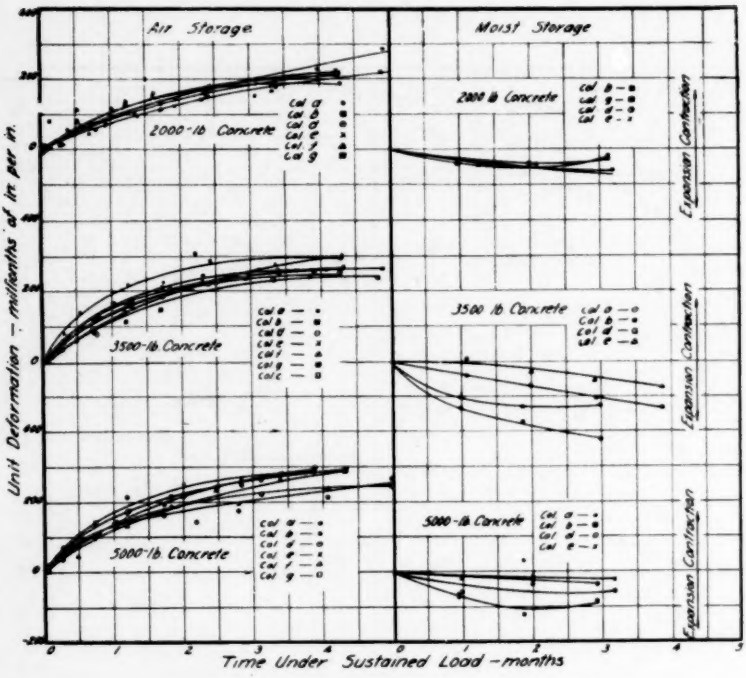

FIG. 9—TIME-DEFORMATION CURVES FOR PLAIN CONCRETE COLUMNS

provided a continuous temperature record over the period of the tests, and a recording psychrometer was installed in the laboratory storage room in January, 1931. Previous to this date, the temperature and humidity records for the laboratory storage room were mainly individual day time observations. In Fig. 10 is shown the record of the average recorded daily temperatures for both dry and moist storage rooms since June, 1930. A similar record of the average relative humidity of the laboratory air is given in the figure. From many observations it is known that the relative humidity of the moist room was maintained at practically 100 per cent, except for short intervals when strain measurements were taken.

The range of temperatures and humidities may be of interest in connection with the curves of Figs. 2 to 9. Knowing from

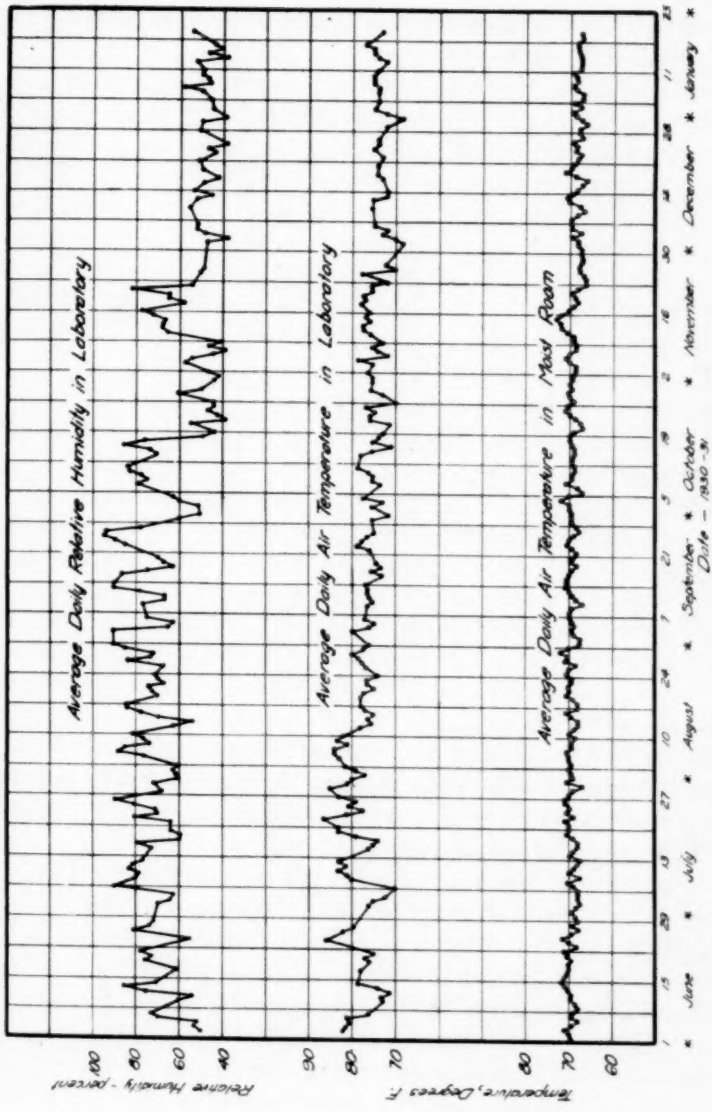

FIG. 10—TEMPERATURE AND HUMIDITY RECORDS FOR PERIOD OF TESTS

Table 1 the date at which loading of any column was started, the trend of temperature and humidity variations thereafter may be traced in Fig. 10, and compared with any unusual variation in the measured strains in the column.

The strains plotted in Figs. 2 to 9 have not been corrected for the effect of temperature changes. For the columns in moist storage such temperature changes were negligible. The air-stored columns are known to have been at very nearly 70 deg. F. when removed from moist storage and placed under spring loading, and subsequent air temperatures of the laboratory are given in Fig. 10; however, it is certain that the effective temperature of any column did not vary as widely or as rapidly as the air temperature of the laboratory. A correction in column strains based on the variation in air temperature might introduce errors as great as if no correction were attempted. It is believed that the uncorrected strains in Figs. 2 to 9 are in no case in error due to temperature effects more than 0.000067 (corresponding to an increase of 10 deg. F. over the initial temperature of the column, and to an indicated tensile stress in the steel of 2000 lb. per sq. in.) and for observations made since August, 1930, the possible error would be much less than this. Any such error is opposite in sense to the effect of shrinkage or flow; if a column has increased in temperature during the loading period, the measured strains attributed to shrinkage and flow are less than the true amount of shrinkage and flow.

The amount of plastic flow, obtained by subtracting the strains of an unloaded column from those of the corresponding loaded one, is unaffected by any temperature changes, since such changes are the same in loaded and unloaded columns.

The strain readings were taken at the time of loading the columns, then generally after 1, 3, 7, 14 and 28 days, and at monthly intervals thereafter. The strain measurements were taken on a total of 16 gage lines, 8 at the middle and 4 each near the top and bottom of column, and included two independent sets of observations. These measurements on a total of 153 columns have required the help of several observers, and in general the strain measurements for any column were taken by two or more observers. The accuracy of the strain gage measurements in this series of tests is therefore considerably less than that obtained

in the other series of the investigation, in which a complete test could be made by one observer in a period of only a few hours, with a definitely established routine for strain measurements.

8. *Elastic Deformations.*—The deformation measured at the time the spring load was applied to any column will be denoted here as an elastic deformation, in contra-distinction to the deformation which resulted from continued loading. Since the concrete stresses were generally well below one-half of the compressive strength, and the steel stress only a small fraction of the proportional limit, it is likely that the strains developed in applying the load were very nearly, if not wholly, elastic. The corresponding steel stresses, which fix the starting point of the curves of Figs. 2 to 8, seem rather erratic from an inspection of the figures. However, a little study of the probable elastic action of the various columns tends to verify the general trend of the measured values. It seems reasonable to assume that the initial stresses due to a load no greater than the design load will be determined by the elastic properties of the two materials; that is, the stresses in concrete and longitudinal steel will be proportional to the respective moduli of elasticity. A large number of extensometer tests of the control cylinders of this series have been made, but not analyzed; however, the average moduli for 10 cylinders, selected at random, of each grade of concrete are as follows:

Designed Strength lb. per sq. in.	Actual Strength lb. per sq. in.	Modulus of Elasticity lb. per sq. in.	Modular Ratio n
2000	2200	2,830,000	10.6
3500	3730	3,800,000	7.9
5000	5460	4,290,000	7.0

These moduli represent the initial slopes of the stress-strain curves, for which the proportional limit was usually 30 to 50 per cent of the ultimate strength. Using 30,000,000 lb. per sq. in. as the modulus for steel, values of the ratio, n, of the moduli for steel and concrete have been computed. This ratio should also represent the ratio of initial steel and concrete stresses. Table 2 gives values of the calculated initial stresses in steel and concrete due to the design loads, based upon the above values of n. The table also shows values of the stresses determined from

TABLE 2. STRESSES IN COLUMNS OF SERIES 3.

The unit stresses, f_s in steel, and f_c in concrete, are in lb. per sq. in. Positive values are compressive stress; negative values, tensile stress.

Nominal Column Design				Loaded Columns							Unloaded Columns	
	Percentage Reinforcement			At Load Application					After 5 Mo.		After 5 Mo.	
Concrete Strength Lb. per sq. in.			Design Load lb.	From Elastic Theory		From Strains			From Strains		From Strains	
	Vertical	Spiral		f_s	f_c	f_s	f_c	f_s/f_c	f_s	f_c	f_c	f_s
Air Storage												
2000	1	2	38000	6900	660	7300	650	11.2	19600	455	5600	— 89
	4	0	37200	5700	535	4300	685	6.3	10800	325	....	....
	4	1	64000	9800	920	9400	940	10.0	16000	665	1200	— 50
	4	2	64000	9800	920	12100	825	14.7	20400	480	1600	— 66
	6	2	90000	12100	1140	12200	1130	10.8	20400*	610	800	— 50
3500	1	0	47200	6750	845	6400	855	7.5	19100	650	....	....
	1	1	52400	7500	940	9800	905	10.8	20000	740	....	....
	1	2	52400	7500	940	7000	945	7.4	21100	725	4300	— 68
	1	2	**67000	9550	1205	11100	1175	9.5	25500	945	....	....
	4	0	55200	6850	6500	6500	870	7.5	16300	470	3400	—140
	4	1	79000	9800	1235	9800	1230	7.9	20000	810	2600	—108
	4	2	79000	9800	1235	10200	1210	8.5	19000	850	2800	—116
	6	0	81200	9100	1145	6000	1335	4.5	13700	850	....	....
	6	1	108600	12050	1540	12400	1515	8.2	24800	730	....	....
	6	2	108600	12050	1540	13100	1470	8.9	21600	930	....	....
	6	2	**75200	8420	1060	8300	1065	7.8	13400	740	....	....
5000	1	2	59400	7650	1080	7300	1080	6.7	22100	850	7200	—114
	4	0	72000	8150	1155	8600	1130	7.6	20000	660	3900	—161
	4	1	96000	10850	1540	10600	1550	10.1	23000	1040	3700	—153
	4	2	96000	10850	1540	9000	1620	5.6	21100	1115	4700	—194
	6	2	130400	13300	1915	15500	1780	8.7	27100	1045	5000	—316
Moist Storage												
2000	4	0	37200	5700	535	4200	595	7.1	10900	325	—3600	149
	4	1	64000	9800	920	9200	945	9.8	8500	975	—2500	104
	4	2	64000	9800	920	9000	950	9.5	9000	950	—1100	45
3500	4	0	55200	6850	860	9600	745	12.9	7500	830	....	....
	4	1	79000	9800	1235	7600	1320	5.8	10100	1210	—1800	75
	4	2	79000	9800	1235	7600	1320	5.8	7900	1310	—3000	124
5000	4	0	72000	8150	1155	6000	1240	4.8	10000	1080	—2400	100
	4	1	96000	10850	1540	11300	1520	7.4	11000	1530	—2200	92
	4	2	96000	10850	1540	9800	1580	6.2	14000	1410	—3700	153

**According to N. Y. City Bldg. Code; all others according to A. C. I. Joint Bldg. Code.
*By extrapolation.

strain measurements. It is of interest that although there are a few rather large discrepancies between the two sets of values, the general trend of the two is quite similar. Furthermore, the average values of the ratio of the measured initial steel and concrete stresses, for columns of the three grades of concrete are 10.0, 8.1, and 7.1, as compared with the above values for n (based on cylinder tests) of 10.6 ,7.9 and 7.0.

The design loads which were computed by the A. C. I. formula for spirally reinforced columns are seen to produce relatively high initial concrete and steel stresses in the columns having a large percentage of longitudinal steel. The ratio of initial concrete stress to the concrete strength is not constant, but in general increases with an increase in percentage of longitudinal reinforcement and decreases with an increase in concrete strength.

Table 2 includes data on the stresses in concrete and steel after a 5 months period of observation for both loaded and unloaded columns. The stress in the unloaded columns may be attributed to the effect of volume changes, while the increase in stress in the loaded columns is evidently due to volume change, plastic flow and change in elastic strain. It is interesting to note that the greatest increase in steel stress due to the three combined effects was 14,800 lb. per sq. in., and in most cases with the air-stored columns the increase was from 6000 to 14,000 lb. per sq. in. For the moist-stored columns, the increase was very much less.

9. *Relation Between Strength of Concrete and Column Deformations.*—For further study of the effect of variations in concrete strengths upon column deformations, curves from Figs. 2 to 8 have been replotted in Fig. 11. Each curve represents the average result from a pair of companion columns and each group of curves represents the results from columns that are alike except for the quality of concrete with which they were made. It is difficult to make comparisons as to the relation between concrete strength and the deformation due to flow and shrinkage because the concrete in the three types of column was subjected initially to neither the same stress nor to the same proportion of the concrete strength in the three cases. Furthermore, for air-stored columns, the stress in the concrete gradually decreased as the concrete deformed so that at the end of the 5-month period the concrete stress had lowered considerably, as shown in Table 2. However, the combined effect upon column action is such that there is no great difference apparent in the time-rate of deformation for the columns made with 2000, 3500 and 5000-lb. concrete. It seems probable that there may be a relation between intensity of concrete stress and the corresponding time-rate of deformation

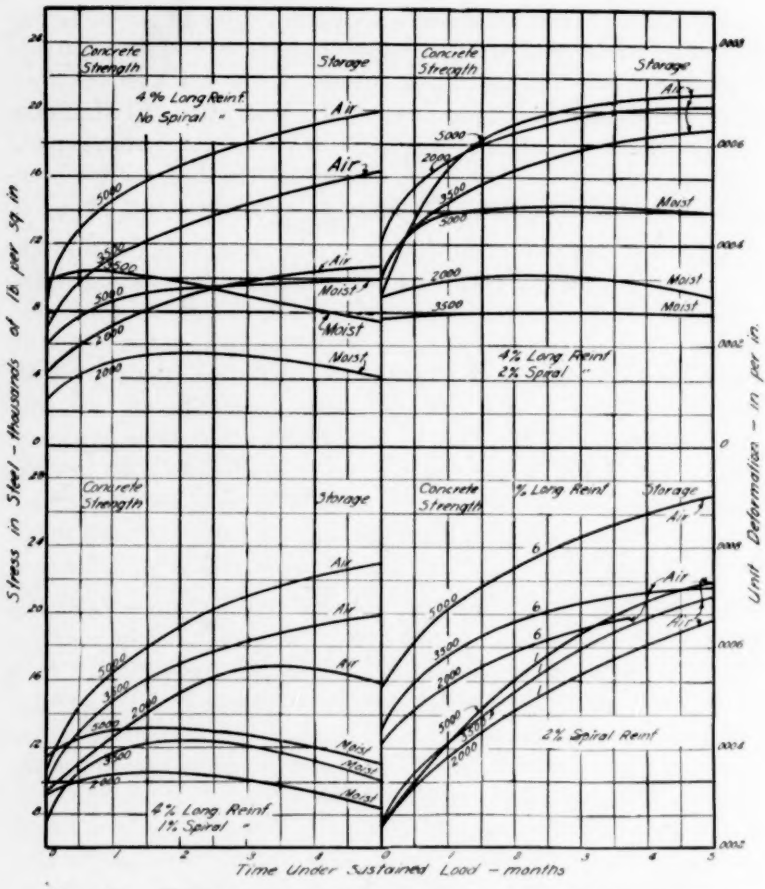

FIG. 11—TIME-DEFORMATION CURVES, VARYING CONCRETE STRENGTHS

at any point on the curves of Fig. 11. Such a relation would be expected to differ for the three grades of concrete.

Fig. 11 also permits a comparison between the deformations found in columns held under load in air and in moist storage. The deformations in the air-stored columns were much larger than those in the moist-cured columns and were increasing at the end of 5 months, whereas in many cases the strains in the moist-stored columns had reached a maximum and were decreas-

ing at the end of the period shown. Such a decrease is also shown by the strain curves for unloaded columns in Figs. 2 to 8 which would indicate that the columns were expanding due to the 5 months storage in the moist room. Since the total deformation of loaded columns is evidently a combination of shrinkage or expansion, plastic flow and elastic deformation produced by the applied load, it seems plausible that the plastic flow after several months of moist storage may be counteracted by the expansion due to the presence of moisture and by an increase in the stiffness of the concrete due to the moist curing. That the elastic strain may decrease due to an increase in modulus of elasticity with age has been shown by Glanville[4] in recent tests of flow of plain concrete under sustained load. This may explain the fact that the flow of plain concrete has been found by Davis[5] and others to be less for specimens in moist storage than for those in dry storage.

10. *Relation Between Percentage of Spiral Reinforcement and Column Deformations*—To permit convenient comparison of time-deformation curves for columns that are alike except for the percentage of spiral reinforcement, Fig. 12 has been drawn. Each pair of curves represents average results for columns with 1.24 and 2.0 per cent of spiral reinforcement, for which the design loads were identical. It was planned to include the results from the tied columns in the comparison, but since the design loads differed from those for spirally reinforced columns, the results did not appear comparable. There is no reason to believe that the presence of spiral reinforcement should have any appreciable effect on the longitudinal strains due to shrinkage, flow or load of the intensity used; this view is supported by the evidence given in Fig. 12. There is no consistent difference in the deformations developed in the presence of the two amounts of spiral, and the differences are of a magnitude which may be explained by errors of observation and differences in the materials and treatment of the columns.

11. *Relation Between Percentage of Longitudinal Reinforcement and Column Deformations.*—Average time-deformation

[4] "The Creep or Flow of Concrete under Load," by W. H. Glanville, Building Research Technical Paper No 12. Department of Scientific and Industrial Research, London, 1930.

[5] "Flow of Concrete Under Sustained Compressive Stress," by R. E. Davis and H. E. Davis, Proc. A. S. T. M., V. 30, Part 2, 1930.

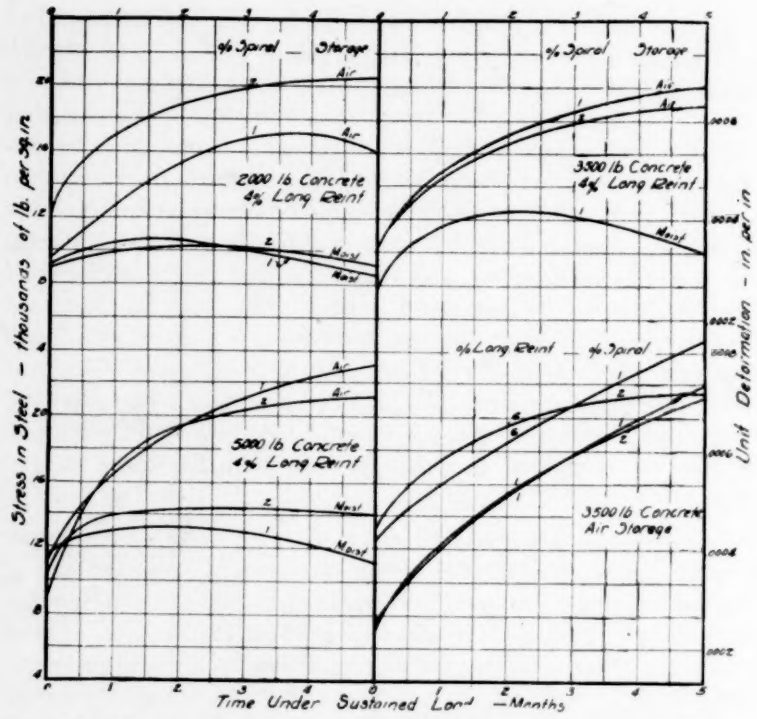

FIG. 12—TIME-DEFORMATION CURVES, VARYING PERCENTAGES OF SPIRAL REINFORCEMENT

curves from Figs. 2 to 8 have been grouped in Fig. 13 to facilitate a study of the effect of variations in amount of longitudinal reinforcement in columns otherwise alike. The three percentages of longitudinal steel were roughly 1.5, 4 and 6. Since all moist-stored columns were made with 4 per cent of longitudinal steel, this comparison is made only between the results from columns stored in air. The figure includes curves for columns made with three grades of concrete and 2.0 per cent spiral reinforcement, also for columns of 3500-lb. concrete with 0, 1 and 2 per cent of spiral reinforcement.

As noted with reference to Table 2, the curves for columns with 6 per cent of longitudinal steel generally start at a higher steel stress than do those for the smaller percentages. This follows from the fact that the A. C. I. formula for spirally rein-

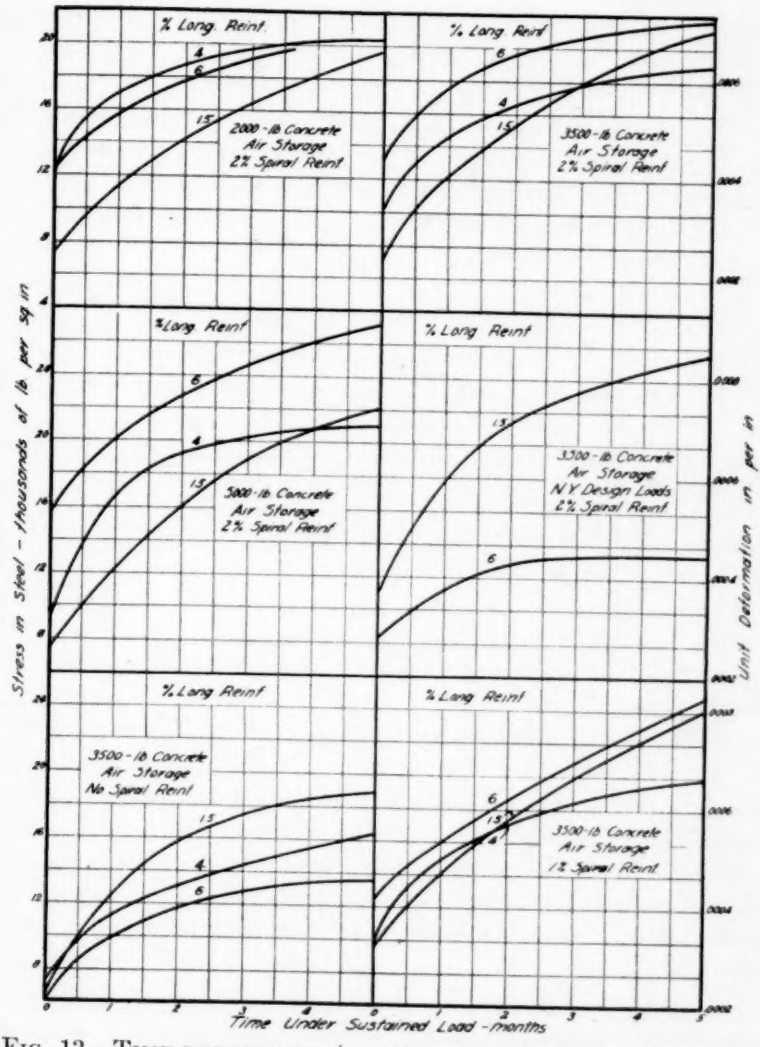

FIG. 13—TIME-DEFORMATION CURVES, VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT

forced columns, used in determining the applied loads, permits an increased working stress in both concrete and steel as the amount of longitudinal reinforcement is increased. The elastic strains, therefore were to some extent fixed by the formula used, and by the modulus of elasticity of the concrete.

The change in deformation, and in the steel stresses, in 5 months was generally least for the columns with 6 per cent reinforcement, slightly more for those with 4 per cent and quite obviously greatest for those with 1.5 per cent. This may be due

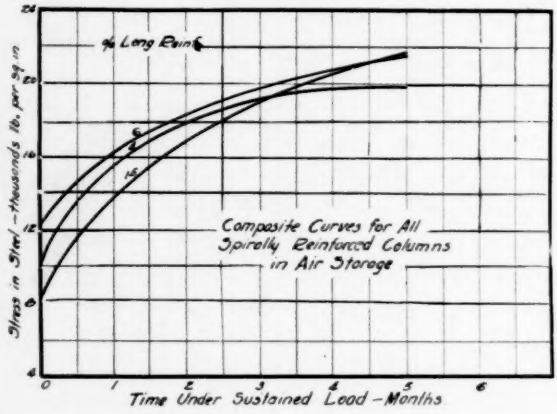

FIG. 14—COMPOSITE CURVES FOR SPIRALLY REINFORCED COLUMNS HAVING VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT

to the fact that a given amount of yielding or flow decreases the concrete stress much more rapidly in columns with a large amount of longitudinal reinforcement than in those with a small amount. There is no marked difference in the appearance of the curves due to variation in concrete strength or in percentage of spiral reinforcement.

To summarize the relation between longitudinal reinforcement and column deformations, Fig. 14 has been drawn by averaging together the results of all air-stored spirally reinforced columns containing each percentage of longitudinal steel, disregarding the concrete strength and the amount of spiral reinforcement. Each of the composite curves of Fig. 14 thus represents the average

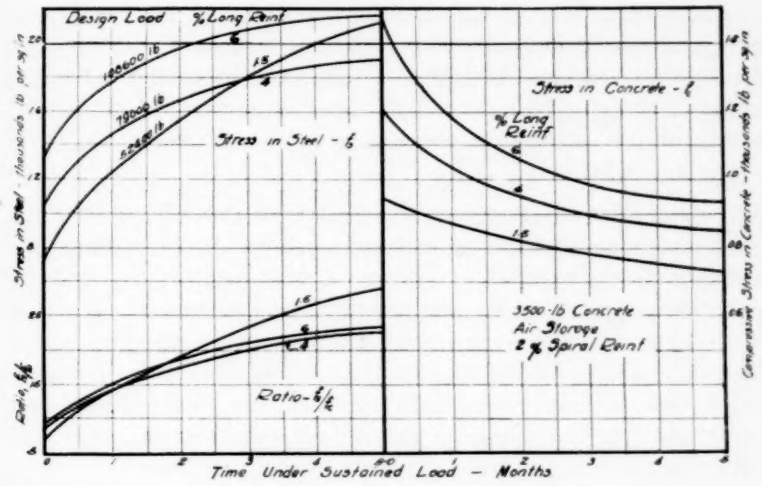

FIG. 15—CURVES SHOWING REDISTRIBUTION OF STRESSES IN CONCRETE AND STEEL DURING A 5-MONTH LOADING PERIOD.

results from ten or more columns. The rate of deformation, and of increase in steel stress during the first two weeks of loading is about equal for the lower two percentages of reinforcement but later on the curve for 4 per cent of reinforcement shows a rapid decrease in rate of deformation as compared to the other two. The curve for 1.5 per cent of reinforcement which started well below the other two, had crossed both of them after 4½ months of loading, and at the end of the 5-month period, shows a much greater rate of increase than the other two curves.

It may be noted that the increase in steel stress during 5 months under load as shown by the composite curves of Fig. 14 were as follows: With 6 per cent of longitudinal reinforcement, 9200 lb. per sq. in.; with 4 per cent, 9700 lb. per sq. in.; and with 1.5 per cent, 13200 lb. per sq. in.

12. *Redistribution of Stresses in Concrete and Steel Due to Inelastic Deformation of Concrete.*—The various time-deformation curves have indicated the increase in the stresses in the longitudinal reinforcement due to shrinkage and flow of the concrete, and it is evident that as the total stress in the steel increased, the total stress in the concrete decreased an equal amount. Table

2 gives a summary of the measured steel stress produced in all columns due to the design load and the increased value of this stress after a 5-months period of loading. From the difference between the applied load and the total stress in the longitudinal steel, the unit stresses in the concrete have also been found, as given in the table.

A conception of the variation in both steel and concrete stresses with time may be aided by reference to Fig. 15. The curves for stress in steel are similar to those of preceding figures, and the corresponding curves showing the concrete stresses at any time during the 5 months have been determined by computation as described above. It is evident that as yielding of the concrete progressed, the concrete stresses fell off rapidly; on the other hand at considerably decreased concrete stresses, yielding was still progressing. Various tests of plastic flow of plain concrete are in agreement on the point that under a constant load the rate of flow gradually decreases with time; if, as in these columns, the applied stress also decreases it would be expected that the plastic flow would progress at a very rapidly diminishing rate. However, tests have been made in which the flow apparently continued for several years, so that it is unsafe to draw any conclusions now as to possible limits in the amount of flow still to be expected.

The lower curves of Fig. 15 show the way in which values of the ratio of steel and concrete stresses varied over the 5-month period. The ratio may be considered equivalent to the modular ratio, n, wherein the modulus of elasticity used is the "sustained modulus of elasticity" and includes the effect of shrinkage, flow and elastic deformation. The curves show no great difference in values of the ratio for the three amounts of reinforcement used, but do show that the values after 5 months of loading average 3 times as great as the initial values. The initial values are in good agreement with the value of n used in computing the elastic distribution of stress in Table 2.

13. *Effect of Initial Strains on Load-Carrying Capacity of Column.*—It is hoped that the completion of this series of tests will furnish information on a number of questions, as yet unanswered, such as: (1) Will the stresses induced in the reinforcing steel reach the yield point of the material, and if so, what will be

the effect upon stresses and deformations, (2) Will the presence of large initial steel strains affect the ultimate strength, or in other words, the factor of safety of the column, and (3) Will the column deformations under continued working loads produce undesirable distortions in a structure?

In the present tests the steel stresses after 5 months are in only one case greater than one-half of the yield point stress for the longitudinal steel, and the rate of yielding is decreasing rapidly. It does not appear probable that the yield point will be reached in any column in the one-year loading period. This is on the assumption that the creep of the reinforcing steel under continued load is negligible at stresses below the yield point as ordinarily defined.

It is known that when a steel bar is stressed beyond the yield point in compression, little lateral force is required to prevent a side-wise buckling if the bar is kept straight. The tests of spirally reinforced columns in the other series of the investigation showed that the longitudinal bars carried load effectively beyond the yield point, though large longitudinal deformations of both concrete and steel took place. The plastic deformation of intermediate grade steel at the yield point is very rapid as compared to the time yields for concrete shown in these tests. Hence it appears that if the yield point of the steel is reached, either due to flow of concrete or to an increase in load, the yielding of the steel will tend to increase the stress in the concrete. Whether, under a constant load, the further yielding of both steel and concrete would be sufficient to bring the spiral reinforcement into action is problematic. However, in the case of the columns which are to be loaded to failure after the end of a year, it seems likely that when the steel reaches its yield point, additional load will be taken by the concrete until it reaches a stress corresponding to the strength of the plain concrete, whereupon the spiral reinforcement will be brought into action. If this is correct, the strength of the column should not be appreciably affected by the initial stresses due to flow and shrinkage.

The deformations in the test columns have not exceeded 0.0009 in. per in., corresponding to a steel stress of 27,000 lb. per sq. in. It does not appear that deformations of this magnitude

should cause any structural difficulties, but if the yield point of the steel were exceeded, the deformations might be of considerable importance.

14. *Summary.*—Little can be stated in the way of conclusions regarding this series of tests, since much of the significance of the results will depend upon the behavior of the columns when tested to failure. It may also be added that in the preparation of this report there has been little time to check up discrepancies in recorded data or to study the results. The following summary may be made as to the foregoing discussion:

1. The stresses in longitudinal reinforcement of columns when initially placed under a working load agree fairly well with the values calculated by assuming the stresses in steel and concrete to be proportional to the respective moduli of elasticity of the two materials.

2. The increase in steel stress in columns held under working loads for 5 months was much greater for columns stored in the air of the laboratory than for those stored in the moist room.

3. The increase in steel stress for the loaded columns was slightly greater for columns with 4 per cent of longitudinal reinforcement than for those with 6 per cent, and much greater for those with 1.5 per cent than for either of the other two.

4. The greatest measured stress in any of the test columns, due to the combined effect of load, shrinkage and flow was 27,100 lb. per sq. in., an increase of 11,600 lb. per sq. in., over the initial stress due to the load alone.

5. The rate of flow does not appear to vary greatly with different grades of concrete or in the presence of different amounts of spiral reinforcement. Any study of the rate of flow is complicated by the fact that the concrete stress decreases as the flow and shrinkage increase.

6. The results thus far obtained in this series of tests indicate that the final results will have a very definite bearing on the question of the proper method of design of reinforced concrete columns.

Readers are referred to the JOURNAL *for June, 1931, for discussion of the Reinforced Concrete Column Investigation of Committee 105. Such discussion should reach the Secretary by May 1, 1931.*

Second Progress Report on Column Tests Made at Lehigh University

BY W. A. SLATER* AND INGE LYSE†

SERIES 3—COLUMNS UNDER LONG TIME LOADS

1. *Introduction.*—The first Progress Report of Committee 105 on Column Investigations was published in the April JOURNAL, 1930.[1] That report gives the program of the investigation and results of the preliminary tests of the materials. The first progress report from Lehigh University on the column tests gives results for Series 1 and 2 and was published in the February, 1931, JOURNAL.[2] The present (second) progress report from Lehigh University gives results for Series 3 on columns subjected to working loads for 20 weeks. Wherever references are made to the results for Series 1 and 2 the information will be found in the first progress report.

2. *Purpose and Program.*—The purpose of Series 3 is to study the effect of sustained working load on the deformation and strength of reinforced concrete columns. The applied working loads were computed according to the formulas given in the present American Concrete Institute Building Code (1928)[3] and the present New York Building Code (1916).[4] The cases to which

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[1]See Progress Report of Committee 105, *Proc. A. C. I.* V. 26, p. 601, 1930.

[2]See Progress Report on Column Tests at Lehigh University, *Proc. A. C. I.* V. 27, p. 677. 1931.

[3] $\frac{P}{A} = \left(300 + (0.10 + 4p) f'_c\right) \left(1 + (n - 1)\, p\right)$

[4] $\frac{P}{A} = f_c \left(1 + (n - 1)\, p + \frac{2 f_s n'}{f_c}\right)$

in which P = total load on column

A = total sectional area within outer circumference of spiral
f'_c = ultimate strength of control cylinders
f_c = allowable working stress in concrete. (Foot note continued next page)

each formula was applied are indicated in Tables 1 and 6. The columns in this series did not conform, however, with either code so far as the thickness of the concrete covering of the reinforcement was concerned. The minimum thickness of this covering should be 1½ in. according to the Institute Code and 2 in. according to the New York Code. All columns in this series had an average thickness of the concrete covering of only ⅛ in. A further deviation from the New York Code was made for the columns having 6 per cent longitudinal reinforcement. Even though the New York Code specifies a maximum of 4 per cent longitudinal reinforcement, the load applied to two of the columns having 6 per cent reinforcement was computed from the formula given by the New York Code. The formulas are designed to be used in connection with the 28-day compressive strength of the control cylinders, whereas in computing the working loads for these tests the strength at the time of application of load (56 days) was used instead.

These rather wide divergences from the limits imposed by the Codes are justified by the fact that this investigation is aimed at determining trends, and the applicability of the principles underlying the formulas, rather than the mere testing of columns conforming in every respect to the limitations, within which the Codes are necessarily surrounded. For example, the shell provided outside the spiral is generally intended as fire protection, but to include such a shell on these test columns would, to a certain degree, defeat one purpose of the investigation which is to determine the strength of the spiral core.

The program of tests for Series 3 is shown in Table 1. This does not differ from the program for Series 3 given in the 1930 *Proceedings*, page 611, but is given in more detail for the sake of clarity. Three types of concrete, designed for cylinder strengths of 2000, 3500, and 5000 lb. per sq. in. at 56 days, respectively, were included. The reinforcement was all of intermediate grade

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f_s = allowable working stress in steel
p = ratio of longitudinal reinforcement
p' = ratio of spiral
n = ratio of moduli

The units used for loads, areas, and stresses are pounds, square inches, and pounds per square inch respectively.

TABLE 1. PROGRAM OF TESTS FOR SERIES 3

Design Strength of Concrete lb. per sq. in.			Reinforcement: Longitudinal per cent			Reinforcement: Spiral per cent			Storage		Method of Test		Number of Columns: Alike	Number of Columns: Total
2000	3500	5000		4		0	1.2	2	Wet	Dry	A. C. I. Load	No Load	2	72
2000	3500	5000	1½		6			2		Dry	A. C. I. Load	No Load	2	24
	3500		1½		6	0	1.2			Dry	A. C. I. Load		2	8
	3500		1½		6			2		Dry	New York Load		2	4
												Total		108

steel. Three percentages of longitudinal reinforcement, approximately 1½, 4, and 6 per cent respectively, were used. The spiral reinforcement amounted to 0 per cent, 1.2 per cent or 2 per cent of the core area of the column. Some of the columns are stored in the moist room, while the others are stored in the testing laboratory during the period of sustained load. Companion columns under no load are stored with the loaded columns.

The columns have an outside diameter of 8¼ in., a core diameter (outside diameter of spiral) of 8 in., and an overall length of 60 in. The longitudinal bars are 60 in. long and have milled ends, thus insuring direct bearing on both steel and concrete.

Six 6 by 12-in. control cylinders were made with each column, three to be tested when the working load was applied to the column, and three to be stored with the column and tested at the time of loading the column to failure. The working load was applied when the columns were 56 days old. This load is to be maintained for one year and then the columns are to be tested to failure. The first group of columns was placed under load on June 12, and the last on August 15, 1930.

The present schedule calls for deformation readings on the columns immediately before and after the application of the working load, then at 1 day, 1 week, 2 weeks, 4 weeks, 12 weeks, 20 weeks, 32 weeks, and 52 weeks, after the application of the load. Up to the present time this schedule has been adhered to. With each set of observations on the loaded columns, a set is taken on the companion columns stored under no load. This report contains data obtained during the first 20 weeks of the test.

3. *Acknowledgment.*—The success of the work has been promoted by the interested cooperation of the men connected with it. Especial mention is made of C. L. Kreidler, Research Fellow, and of C. C. Keyser, Laboratory Assistant, for their valuable assistance in carrying out the tests and preparing the data for publication.

4. *Proportioning and Strength of Concrete.*—The cement and aggregates used in this series came from the same supply as that used in Series 1 and 2. The mixes used for the different strengths

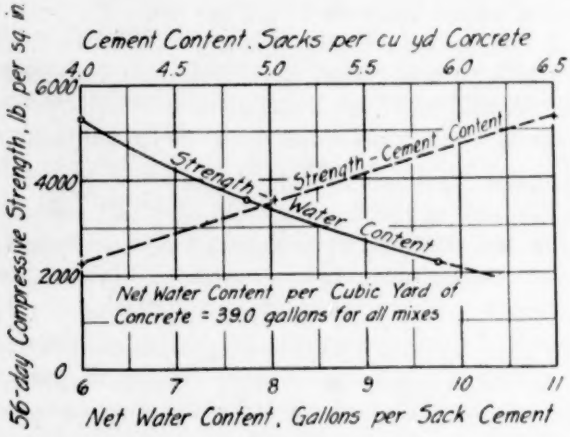


FIG. 1—RELATION BETWEEN STRENGTH AND WATER CONTENT AND BETWEEN STRENGTH AND CEMENT CONTENT FOR CONCRETE USED IN SERIES 3

of concrete were the same as those used in Series 2. Two batches of concrete were required for each column and its six control cylinders. Four minutes mixing was used, one minute dry, and three minutes wet. The two batches were dumped into a metal container and mixed together by hand before placing in the column mold. Table 2 shows the uniformity in strength and slump. The uniformity throughout the series is good, although nearly nine weeks elapsed between the making of the first and the last columns designed to have concrete of the same strength.

TABLE 2. DATA OF CONCRETE OF SERIES 3

Date of Make	Mix by Weight	Water-Cement Ratio by Weight	Slump in.	Strength at 56 Days lb. per sq. in.
Designed for Compressive Strength of 2000 Lb. per Sq. In.				
1930				
4–21	1 : 3.42 : 5.13	.864	3¼	2140
4–23	1 : 3.42 : 5.13	.864	2¾	2180
5– 5	1 : 3.42 : 5.13	.864	2¾	2240
5–19	1 : 3.42 : 5.13	.864	3	2210
5–21	1 : 3.42 : 5.13	.864	3	2240
6– 2	1 : 3.42 : 5.13	.864	2½	2270
6– 4	1 : 3.42 : 5.13	.864	3	2320
6–18	1 : 3.42 : 5.13	.864	2½	2290
Designed for Compressive Strength of 3500 Lb. per Sq. In.				
4–17	1 : 2.65 : 4.00	.686	4¼	3380
4–18	1 : 2.65 : 4.00	.686	4½	3330
4–25	1 : 2.65 : 4.00	.686	4½	3540
4–28	1 : 2.65 : 4.00	.686	4¼	3350
5– 8	1 : 2.65 : 4.00	.686	5¼	3560
5–12	1 : 2.65 : 4.00	.686	5	3730
5–16	1 : 2.65 : 4.00	.686	5¼	3780
5–23	1 : 2.65 : 4.00	.686	4¼	3700
5–26	1 : 2.65 : 4.00	.686	4½	3870
6– 6	1 : 2.65 : 4.00	.686	4¼	3900
6– 9	1 : 2.65 : 4.00	.686	4	3670
6–16	1 : 2.65 : 4.00	.686	3¾	3240*
Designed for Compressive Strength of 5000 Lb. per Sq. In.				
4–30	1 : 1.98 : 2.96	.531	6	5180
5– 2	1 : 1.98 : 2.96	.531	5	4600†
5–14	1 : 1.98 : 2.96	.531	5¾	5560
5–28	1 : 1.98 : 2.96	.531	4¾	5670
5–30	1 : 1.98 : 2.96	.531	4¾	5600
6–11	1 : 1.98 : 2.96	.531	5½	5120
6–13	1 : 1.98 : 2.96	.531	5½	5270
6–20	1 : 1.98 : 2.96	.531	4½	5050

*Eccentric Loaded. †Due to split head of testing machine.

Table 3 gives the summarized data of the concrete. A good agreement was found between the strength designed and that secured. The water content was 39.0 gal. per cu. yd. of concrete for all mixes. This produced a placeable concrete and only a small variation was found in the consistency as measured by the slump. Fig. 1 shows the relation between the strength of the concrete and the water-cement ratio used in the mix. The strength-water-content relation was a smooth curve very similar to that obtained for Series 2. The relation between cement content and strength is given by the straight, broken line. This diagram confirms the results of former tests which have indicated that with the same aggregates and a constant water content per cu. yd. of concrete, the slump was approximately constant and

TABLE 3. SUMMARIZED DATA OF CONCRETE OF SERIES 3

Strength of Concrete at 56 Days lb. per sq. in.		Mix by		Net Water Content* Gallons per		Cement Content Sacks pe. cu. yd. Concrete	Average Slump in.
Designed	Obtained	Weight	Absolute Volume	Sack Cement	Cu. Yd. Concrete		
2000	2230	1 : 3.42 : 5.13	1 : 4.08 : 6.11	9¾	39.0	4.00	2¾
3500	3590	1 : 2.65 : 4.00	1 : 3.16 : 4.77	7¾	39.0	5.02	4½
5000	5260	1 : 1.98 : 2.96	1 : 2.36 : 3.53	6	39.0	6.50	5¼

*Assumed 0.7 per cent absorption by weight of dry aggregates.

the strength increased in proportion to the increase in the cement content.

5. *Modulus of Elasticity of Concrete.*—The modulus of elasticity was determined on one control cylinder from each column. The apparatus used was the same as that described in the report of Series 2. The results obtained were averaged for each type of concrete used, and these averages are plotted in Fig. 2. The slope of the tangent at a stress of 500 lb. per sq. in. was taken as the modulus of elasticity of the concrete.

6. *Reinforcement.*—The longitudinal reinforcement consisted of intermediate grade deformed steel bars and came from the same supply as that described in Series 2. The bars were 22 ft. long when received. At the laboratory they were cut into four 5 ft. lengths and a 2-ft. test coupon. The coupons from each set of four bars were tested for yield-point stress and ultimate strength. The summary of these tests is given in Table 4. Both yield-point stress and the ultimate strength of the coupons (based upon nominal areas) varied considerably. The variation in weights of bars of the same nominal size indicates that the principal variation in strength was probably due to the variation of the cross-section of the bars, rather than to difference in quality of the steel.

The lateral reinforcement consisted of intermediate grade wire spirals. The spirals had an outside diameter of 8 in., and an overall length of 59 in. Three extra turns were provided at each end of the spiral for anchorage. A No. 5 rod (nominal diameter 0.207 in.) having a 1.35-in. pitch gave 1.2 per cent lateral reinforcement and a ¼-in. rod having a 1.20-in. pitch gave 2 per cent lateral reinforcement. An identified test coupon accompanied

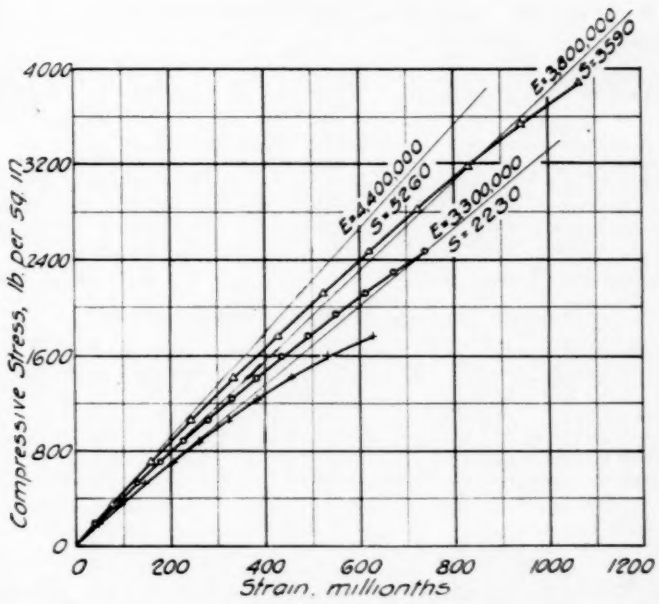


FIG. 2—AVERAGE STRESS-STRAIN DIAGRAM FOR CONTROL CYLINDERS OF SERIES 3

each spiral as received. The average tensile strengths of these coupons were 83,200 and 73,400 lb. per sq. in. for the No. 5 and the ½-in. rods respectively (see Table 5). It will be noted that the ultimate strength of the No. 5 rod was considerably above that of the ¼-in. rod and that of the longitudinal reinforcement.

As described in the report for Series 2, the reinforcement for columns having both longitudinal and spiral reinforcement was assembled into a unit before being placed in the mold.

In the columns having longitudinal reinforcement but no lateral reinforcement, the bars were held in position by tying to five No. 10 wire hoops equally spaced. These hoops did not provide appreciable lateral reinforcement, both because of the large spacing and because in each hoop the ends of the wires were merely hooked together.

7. *Placing and Curing of Concrete.*—The placing and curing of the concrete were the same as those used in Series 2. When

TABLE 4. SUMMARY OF TESTS OF LONGITUDINAL BARS

Nominal Size of Bar	Number of Coupons Tested	Yield Point Stress lb. per sq. in.	Ultimate* Strength lb. per sq. in.
½-in. square	144	43,000	66,480
½-in. round	18	43,490	74,490
⅝-in. round	18	43,780	72,380
¾-in. round	18	44,930	68,980

*Based on nominal areas.

TABLE 5. RESULTS OF TESTS OF SPIRAL COUPONS

Nominal Size of Bar	Number of Coupons Tested	Average Tensile Strength* lb. per sq. in.
No. 5 (0.207-in. diam.)	29	83,200
¼-in. diam.	60	73,400

*Based on nominal areas.

four columns of the same mix and reinforcement were concreted on the same day, two were for test under sustained working load. and the two companion columns were for storage under no load, Some of the columns tested under sustained load did not have companion columns stored under no load.

The columns and their control cylinders were cured in the moist room for 56 days.

8. *Working Loads.*—From the standpoint of interpreting the data it would have been desirable to use such a test load as would produce the same initial strain in all columns of the same concrete strength. It was not possible, however, to do this and at the same time to use as test loads the working loads computed from the building codes referred to. The amount of variation of the strain in the concrete at the application of the load is indicated in Table 6.

The formula given by the American Concrete Institute Code for spirally reinforced columns was used in determining the test loads for the columns without, as well as for those with spiral reinforcement. This could be done consistently because there is no term in the formula which represents the amount of spiral re-

TABLE 6. TAUBLATION OF STRAINS AND STRESSES

Column No.		Design Str. of Concrete, lb. per sq. in.	Reinf Per Cent		Total Load lb.	Conditions at Loading			Conditions at Twenty Weeks					
									Effect of Shrinkage and Flow			Effect of Shrinkage		
							Stress, Lb. per sq. in.			Stress, lb. per sq. in.			Stress, lb. per sq. in.	
Loaded	Not Loaded		Long.	Spiral		Strain Millionths	Concrete	Steel	Strain Millionths	Concrete	Steel	Strain Millionths	Concrete	Steel
										**	**		**	**
Wet Storage														
9, 10	11, 12	2,000	4	0	64,400	421	810	12,630	627	550	18,810	40	—50	1,200
17, 18	19, 20	3,500	4	0	79,500	384	1,170	11,520	576	930	17,280	40	—50	1,200
25, 26	27, 28	5,000	4	0	96,500	396	1,510	11,880	604	1,250	18,120	17	—20	510
53, 54	55, 56	2,000	4	1.2	64,400	354	890	10,620	524	680	15,720	88	—110	2,640
61, 62	63, 64	3,500	4	1.2	79,500	414	1,130	12,420	585	920	17,550	36	—40	1,080
69, 70	71, 72	5,000	4	1.2	96,500	414	1,480	12,420	634	1,210	19,020	32	—40	960
77, 78	79, 80	2,000	4	2	64,400	341	910	10,230	522	680	15,660	42	—50	1,260
85, 86	87, 88	3,500	4	2	79,500	358	1,200	10,740	542	970	16,260	18	—20	540
93, 94	95, 96	5,000	4	2	96,500	441	1,450	13,230	612	1,240	18,360	26	—30	780
Dry Storage														
1, 2		3,500	1½	0	49,000	298	850	8,940	1,206	420	36,180			
3, 4		3,500	6	0	109,000	488	1,380	14,640	1,086	250	32,580			
5, 6	7, 8	2,000	4	0	64,400	316	940	9,480	972	130	29,160	168	—210	5,040
13, 14	15, 16	3,500	4	0	79,500	369	1,190	11,070	1,170	190	35,100	241	—300	7,230
21, 22	23, 24	5,000	4	0	96,500	447	1,440	13,410	1,151	570	34,530	253	—310	7,590
29, 30	31, 32	2,000	1½	2	38,400	232	670	6,960	906	340	27,180	194	—90	5,820
33, 34		3,500	1½	1.2	49,000	312	840	9,360	1.053	490	31,590			
35 36		3,500	6	1.2	109,000	441	1,470	13,230	996	420	29,880			
37, 38	39, 40	3,500	1½	2	49,000	246	870	7,380	1,027	500	30,810	333	—160	9,990
41, 42	43, 44	5,000	1½	2	60,300	345	1,050	10,350	1,123	680	33,690	375	—180	11,250
45, 46		3,500	1½	2	67,500*	390	1,180	11,700	1,422	690	42,660			
47, 48		3,500	6	2	82,300*	370	1,040	11,100	959	—80	28,770			
49, 50	51, 52	2,000	4	1.2	64,400	342	910	10,260	974	120	29,220	212	—260	6,360
57, 58	59, 60	3,500	4	1.2	79,500	396	1,150	11,880	1,038	360	31,140	268	—330	8,040
65, 66	67, 68	5,000	4	1.2	96,500	403	1,520	12,090	1,049	690	31,470	249	—310	7,470
73, 74	75, 76	2,000	4	2	64,400	355	890	10,650	1,022	60	30,660	206	—260	6,180
81, 82	83, 84	3,500	4	2	79,500	380	1,170	11,400	1,038	360	31,080	225	—280	6,750
89, 90	91, 92	5,000	4	2	96,500	411	1,490	12,330	1,045	700	31,350	299	—370	8,970
97, 98	99, 100	3,500	6	2	109,000	490	1,380	14,700	1,048	320	31,440	195	—370	5,850
101, 102	103, 104	2,000	6	2	90,500	485	990	14,550	982	50	29,460	144	—270	4,320
105, 106	107, 108	5,000	6	2	130,600	535	1,750	16,050	1,125	630	33,750	239	—450	7,170

*Load according to New York Building Code, remainder according to A. C. I. Building Code
**No sign indicates compression, minus sign indicates tension.

inforcement, and it was desirable to do it because it permitted a direct comparison of the deformations under the same load for columns without as for those with spiral reinforcement.

9. *Applying Working Load.*—At the age of 56 days the columns were removed from the moist room. The design load was applied to the columns scheduled to be held under load for one year. The two columns of the same mix and the same reinforcement were placed one on top of the other so that one loading rig served for

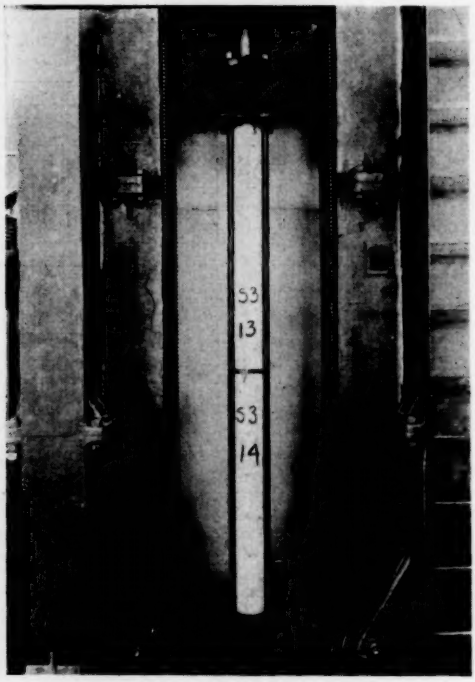


FIG. 3—APPLYING WORKING LOAD TO COLUMNS

both columns. The loading rig and the method used in applying the load to two columns simultaneously are shown in Fig. 3. The load was applied through helical springs held between two loading plates at the upper end of the rig. Steel rods, 1½ in. in diameter and 12½ ft. long, transmitted the load from the springs to the columns. In order to secure a good distribution of the load, planed steel bearing plates were used at the ends of the columns and pieces of beaver board were inserted between the bearing plates and the loading plates. A steel plate, planed on both faces, was inserted between the columns.

The rods and loading plates were donated by the Bethlehem Steel Co. of Bethlehem, Pennsylvania.

FIG. 4—COLUMNS OF SERIES 3 UNDER SUSTAINED LOAD IN DRY STORAGE

The load was applied to the columns in an 800,000-lb. vertical-screw testing machine. The two columns and the loading rig were assembled on the table of the machine and the design load applied by bringing the head of the machine down upon the upper plate, which bore upon the springs. The length of the springs in their compressed condition under the design load was measured to the nearest 0.001 in. A small excess load was then applied to compensate for the elongation of the rods and deflection of the plates to be expected upon the raising of the head of the testing machine. The nuts were then tightened on the rods, after which all the load exerted by the testing machine was released. The length of the springs was then remeasured, and if it did not agree with that which had been found under the design load, successive trials were made until agreement was secured.

Strain gage readings were taken on all columns immediately before and after the application of the load. The loaded columns and their unloaded companion columns were then stored as scheduled, either wet or dry. The columns stored dry are shown in Fig. 4.

10. *Maintaining the Working Load on Columns in Dry Storage.* —The springs are holding the load fairly well. However, there

is a slight falling off of load, due to the deforming of the columns, and also to permanent set in the springs. The length of the springs was measured after each series of strain gage readings. After 12 weeks under load it appeared that the falling off of the load was of sufficient importance to necessitate readjustment of the springs. The loading rig containing the columns was placed in the testing machine and the correct working load re-applied. The method used for applying the load was also used for the readjustment. The average decrease in 12 weeks was about 7 per cent of the correct working load. Of this, about 4 per cent was due to the deformation of the columns and 3 per cent due to permanent set in the springs.

The columns stored in the moist room showed so slight an increase in deformation during the time under load that it has been unnecessary to readjust the loads.

11. *Data Obtained During the First Twenty Weeks Under Load.* —For purposes of reference the average measured unit deformations for each pair of companion columns of Series 3, are given at the end of this report in Figs. 15 to 44. The agreement between companion columns was very good as is illustrated in Fig. 5. It is not feasible in this report, to include similar results for all columns.

There were 20 gage lines on each column for the measurement of deformation in the steel and 20 for the concrete. Two readings were taken on each gage line with each observation. With some exceptions, the results of 80 readings (40 on each of two columns) therefore were averaged to give each plotted point in Figs. 15 to 44.

The agreement between the deformations observed in the concrete and those in the steel was very good. The most important exception noted is that of columns 81 and 82 (Fig. 38) which show a difference in deformation of about 50 millionths.

The average of longitudinal strains measured at opposite ends of any diameter has been found equal (or nearly so) to the average of the longitudinal strains at opposite ends of the diameter lying at right angles to, and in the same horizontal plane with the former diameter. This applied not only to the elastic strains

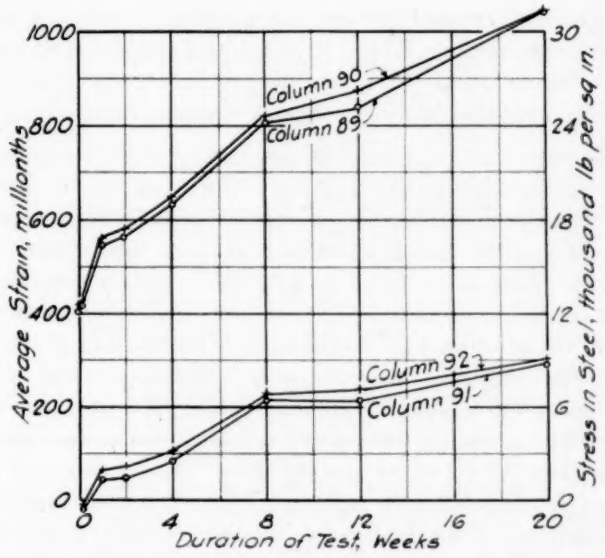


FIG. 5—AGREEMENT BETWEEN DEFORMATION OF COMPANION COLUMNS

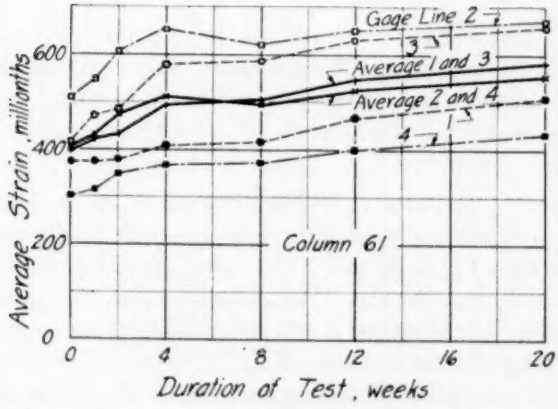


FIG. 6—ILLUSTRATION OF CONSERVATION OF PLANE SECTION

measured at the time of application of load, but also to the shrinkage and increase in deformation during an extended period. An illustration of the relations here pointed out is given in Fig. 6. In the working up of the strain data reliance was placed upon this relation for the detection of error in the computations.

All the columns in dry storage generally showed large increase in deformation during the first two to four weeks under sustained load. From then on the rate of increase in deformation became smaller and more nearly constant as time progressed. The companion columns under no load showed a steady shrinkage for the first 12 weeks of test, but from then on there was little increase in deformation due to shrinkage. The shrinkage deformations of the unloaded columns were subtracted from the deformations of the companion columns under load when both were present. The resulting time-deformation curves so corrected are shown in Fig. 15 to 44 as broken lines. Each corrected curve follows approximately a straight line from about two weeks after the application of the load until the end of the twenty weeks period.

The columns in wet storage showed a much smaller increase in deformation during the loading period than did those in dry storage. The deformation of the unloaded companion columns was quite small throughout the test. As with the columns stored dry each corrected deformation curve for the loaded columns stored wet followed approximately a straight line from about two weeks after the beginning until the end of the twenty weeks under load. It is noted that the increase in deformation due to the load was much smaller for the columns in wet storage than for those in dry storage.

12. *Effect of Variation in Strength of Concrete on Deformation.*—Figs. 7 and 8 give the unit deformations under load for columns made with concrete designed to have strengths of 2000, 3500, and 5000 lb. per sq. in. In Fig. 7 the solid lines and the broken lines represent the deformations of columns having 6 per cent and 1½ per cent longitudinal reinforcement. The spiral reinforcement was 2 per cent in all columns represented in Fig. 7. Each curve in Fig. 8 represents the measured strains for the columns with concrete of a certain designed strength. All the columns rep-

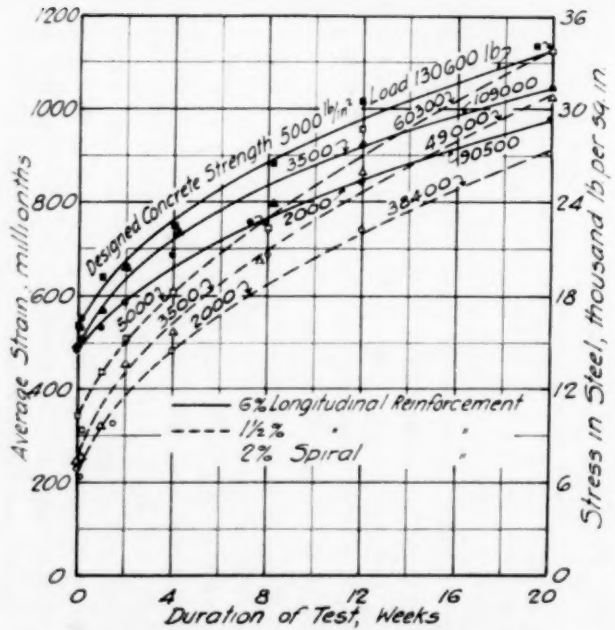


FIG. 7—EFFECT OF VARIATION IN STRENGTH OF CONCRETE ON INCREASE IN DEFORMATION OF COLUMNS HAVING 2 PER CENT SPIRAL REINFORCEMENT AND 1½ PER CENT AND 6 PER CENT LONGITUDINAL REINFORCEMENT, DRY STORAGE

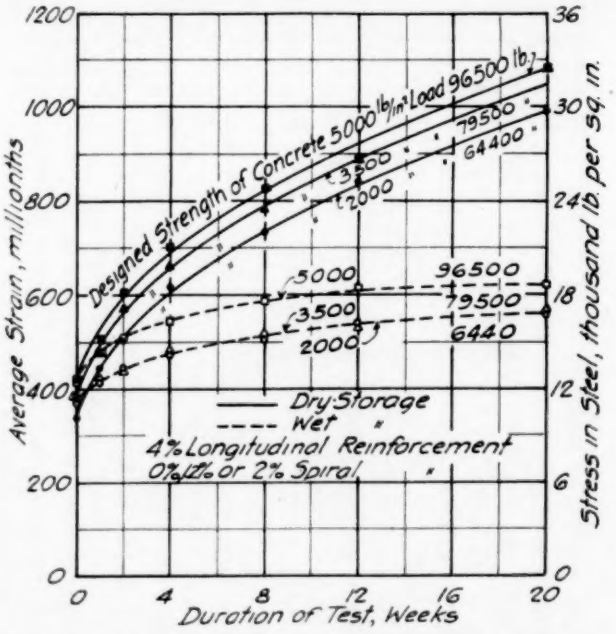


FIG. 8—EFFECT OF VARIATION IN STRENGTH OF CONCRETE ON INCREASE IN DEFORMATION OF COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT AND 0 PER CENT, 1.2 PER CENT, OR 2 PER CENT SPIRAL REINFORCEMENT

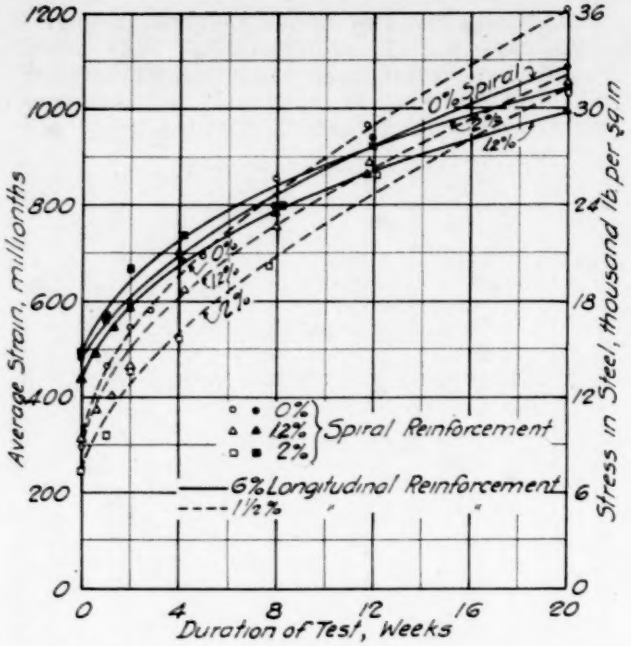


FIG. 9—EFFECT OF VARIATION IN PERCENTAGE OF SPIRAL REINFORCEMENT ON INCREASE IN DEFORMATION OF COLUMNS. DESIGNED STRENGTH OF CONCRETE 3500 LB. PER SQ. IN., 1½ PER CENT AND 6 PER CENT LONGITUDINAL REINFORCEMENT, DRY STORAGE

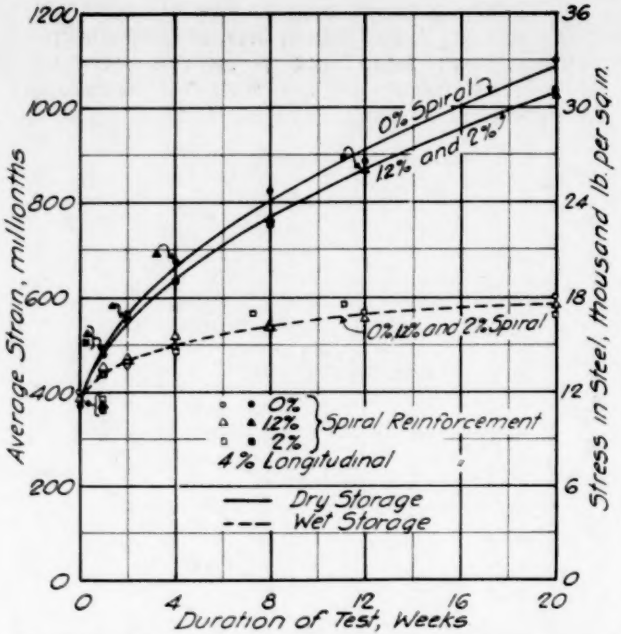


FIG. 10—EFFECT OF VARIATION IN PERCENTAGE OF SPIRAL REINFORCEMENT ON INCREASE IN DEFORMATION OF COLUMNS HAVING 4 PER CENT LONGITUDINAL REINFORCEMENT, DESIGNED STRENGTH OF CONCRETE 2000, 3500, AND 5000 LB. PER SQ. IN.

resented had 4 per cent of longitudinal reinforcement. The spiral reinforcement was either 0, 1.2, or 2 per cent. It is later shown that the strains were practically independent of the amount of the spiral reinforcement, and this fact is used as the justification for averaging the strains indiscriminately regardless of the amount of the spiral reinforcement. The solid lines represent the columns in dry storage and the broken lines those in wet storage. The most striking feature of these figures is that the strain curves for a given type of storage are substantially parallel. That is, within the range of these tests the rate of increase in deformation was everywhere practically independent of the strength of the concrete and of the initial stress. This comment applies to the columns held under load in wet storage as well as to those in dry storage. However, the amount of increase in deformation was very much smaller for the columns loaded in wet, than for those loaded in dry storage. It is probable that the reason for the apparent lack of dependence of the rate of increase in deformation upon the strength of the concrete is the fact that under the code used for determining the working load the concretes of the higher strengths required correspondingly greater loads. In other words, the effect of the higher strengths apparently was offset by the corresponding higher stresses.

13. *Effect of Amount of Spiral Reinforcement on Deformation.*—Fig. 9 and 10 give the time-deformation curves for columns with 0, 1.2, and 2 per cent spiral reinforcement. For all of the columns with a given percentage of longitudinal reinforcement and of a given concrete strength the same load was applied, regardless of the percentage of spiral used. At the time of applying the load the deformations were greatest in some instances for the columns having no spiral and in others for those having 1.2 or 2 per cent of spiral. This indicates that slight variations in the modulus of elasticity of the concrete had more effect on the amount of initial deformation than did the presence or absence of spiral reinforcement. In Fig. 9 the deformations of columns made of concrete having a designed strength of 3500 lb. per sq. in. are shown for 0, 1.2, and 2 per cent of spiral reinforcement and for 1½ and 6 per cent of longitudinal reinforcement. The solid lines represent the columns having 6 per cent and the broken lines those having 1½

per cent longitudinal reinforcement. All columns in this figure were stored dry. Fig. 10 shows the deformations obtained on columns having 4 per cent of longitudinal reinforcement. In Fig. 10 each point represents the average of the deformations of the columns having a given percentage of spiral reinforcement, regardless of the strength of the concrete, which varied between 2000, 3500, and 5000 lb. per sq. in. Since the same variations in strength of concrete enter for one curve as for another in this figure, the curves may be compared in studying the effect of the amount of spiral reinforcement on the deformations. The solid lines represent the deformations of the columns stored dry, and the broken lines those of the columns stored wet.

Within the range of accuracy of the results, except for the columns without spiral reinforcement, the time-deformation curves were either identical or parallel, depending on whether the deformations immediately after applying the load were equal or not. The results indicate therefore, that the rate of increase in deformation was practically independent of the percentage of spiral reinforcement.

14. *Effect of Variation in Longitudinal Reinforcement on Deformation.*—In Fig. 11 are shown the deformations which occurred in the columns during the first 20 weeks of test. To eliminate the irregularities caused by the small number of specimens, an average curve was prepared for all the columns having 1½ per cent longitudinal reinforcement, regardless of the percentage of the spiral, or the strength of the concrete. Similar average curves for 4 per cent and 6 per cent longitudinal reinforcement were prepared and all three are shown in Fig. 11.

The greater the percentage of longitudinal reinforcement, the greater were the applied loads and the initial deformation. This is because under the American Concrete Institute Code (which was used for determining the test loads) an increase in longitudinal reinforcement calls for an increase in the working stress in the concrete. The figure indicates that the rate of increase in deformation with the time under load was greatest for the smallest percentage of longitudinal reinforcement and the least for the largest percentage.

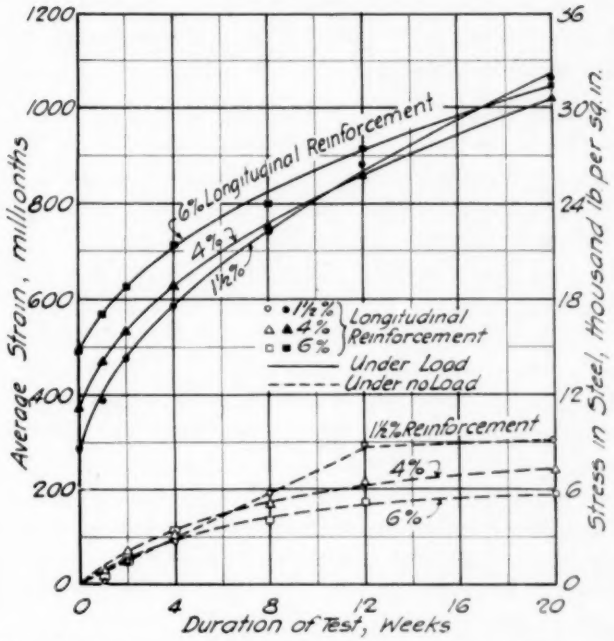


FIG. 11—AVERAGE DEFORMATION OF COLUMNS HAVING VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT, DRY STORAGE

The broken lines in the lower part of Fig. 11 show the effect of the variation in percentage of longitudinal reinforcement upon the shrinkage of the columns under no load. For the first four weeks of the test, the shrinkage was nearly the same for the three different percentages of reinforcement. From then on, the shrinkage of the columns with the largest percentage of reinforcement was much smaller than the shrinkage of the columns having smaller percentages. After 20 weeks the shrinkage deformations of the columns having 6, 4, and 1½ per cent longitudinal reinforcement were 190, 240, and 300 millionths, respectively. If the shrinkage were subtracted from the companion columns under sustained load, the time-deformation curves would become nearly parallel except for slightly smaller vertical distance between the curves at the end of 20 weeks than at the beginning of the test. Evidently the resistance offered by the added reinforce-

ment offset the effect of the greater initial concrete stress on the rate of deformation for columns having the greater percentages of longitudinal reinforcement.

15. *Comparison Between New York and American Concrete Institute Building Codes.*—Fig. 12 shows the deformations of the two groups of companion columns loaded in conformity with the New York and the American Concrete Institute Building Codes. The solid lines represent the deformations of the columns loaded according to the New York Code, and the broken lines those loaded according to the Institute Code. It is noted that for the former the initial concrete stresses for the columns having 6 and 1½ per cent longitudinal reinforcement respectively, were practically equal. The rate of increase in deformation was much greater for the columns having 1½ per cent reinforcement than for those having 6 per cent. That is, the deformation curves departed continuously from each other as time progressed. At the end of 20 weeks the stress in the steel of the columns having 1½ per cent longitudinal reinforcement was about 14,000 lb. per sq. in. (48 per cent) greater than that for the columns having 6 per cent reinforcement.

The broken lines represent the deformations of the columns loaded according to the Institute Code. Here the curves approached each other as time progressed, and at the end of 20 weeks the deformations of the columns having 1½ and 6 per cent longitudinal reinforcement were nearly equal, even though the columns with 6 per cent had about twice as large initial deformation as did those with 1½ per cent reinforcement.

If, as is present practice, the stress in the longitudinal reinforcement is an important criterion in the design of concrete columns, the American Concrete Institute Building Code gave a better basis for designing reinforced concrete columns than did the present New York Building Code. There are, however, indications that the strength of a reinforced column is equal to the sum of the strength of the concrete and the yield-point strength of the longitudinal reinforcement. If this were to be established as the criterion for the design of columns, the importance of maintaining a fixed stress in the longitudinal reinforcement, regardless of its percentage, would become of less significance.

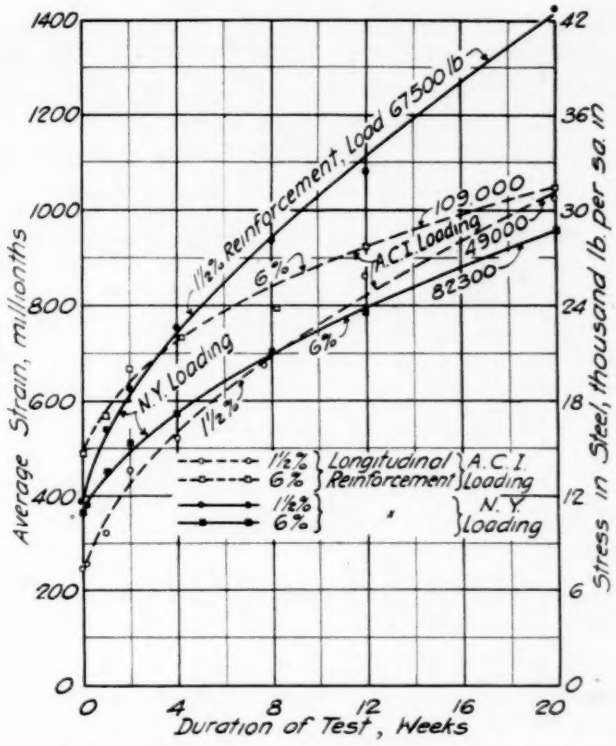


FIG. 12—DEFORMATION OF COLUMNS LOADED IN CONFORMANCE WITH THE NEW YORK AND THE AMERICAN CONCRETE INSTITUTE BUILDING CODES, DESIGNED STRENGTH OF CONCRETE 3500 LB. PER SQ. IN., 2 PER CENT SPIRAL REINFORCEMENT, DRY STORAGE

16. *Shrinkage of Columns Stored Dry.*—Fig. 13 shows the shrinkage for columns stored in the air of the laboratory. The shrinkage of columns having 1½, 4, and 6 per cent longitudinal reinforcement respectively, has been shown for each strength of concrete. Each point represents the average for all gage lines on all columns having concrete of the same strength and the same percentage of longitudinal reinforcement, regardless of the percentage of spiral reinforcement. It is noted that up to four weeks dry storage no marked difference was found for columns of different percentages of longitudinal reinforcement. From then on, however, the shrinkage for the columns having 1½ per cent

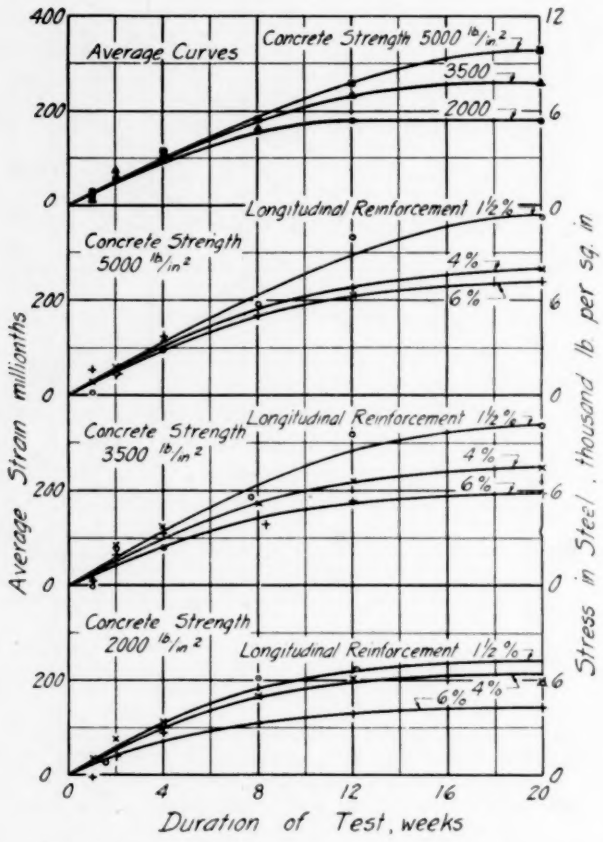


FIG. 13—SHRINKAGE OF COLUMNS WITH VARIOUS PERCENTAGES OF LONGITUDINAL REINFORCEMENT AND OF DIFFERENT STRENGTHS OF CONCRETE

was greater than that for columns having 4 per cent, and the latter in turn was greater than the shrinkage for columns having 6 per cent longitudinal reinforcement. With columns having a given percentage of longitudinal reinforcement, the greater the strength of the concrete the greater was the shrinkage. The effect of the strength of the concrete upon the shrinkage is shown

most directly in the top part of the diagram. Here the average shrinkage for each strength of concrete is shown, regardless of the percentage of longitudinal reinforcement. It was found that after the columns had been under test for more than four weeks the shrinkage was consistently greater with the concretes of the greater strengths.

17. *Summary of Deformations and Stresses*—Table 6 summarizes the deformations and the resulting stresses at the time of application of load and after 20 weeks of test, for the columns included in this series. The table includes the data of both sets of tests, those in which the columns had been loaded with the design load for 20 weeks, and those in which the columns were not loaded but in which the strains due to shrinkage had been measured. It includes also both sets of storage conditions, wet at a temperature of 70 deg. F., and dry at room temperature, which varied between 60 deg. F. and 95 deg. F. during the 20 weeks.

An illustration of how the stress in the steel increased and that in the concrete decreased during the time under sustained load is given in Fig. 14. The curves in the lower half of the diagram represent columns having a concrete strength of 5000, and those in the upper half of 2000 lb. per sq. in. In all these columns the reinforcement consisted of 2 per cent spiral and 1½ or 6 per cent longitudinal. The solid lines represent the stresses in the steel and the broken lines those in the concrete.

It is seen from Table 6 and Fig. 14 that the decrease in stress in the concrete due to the increase in deformation, was greater for the columns with high than for those with low percentages of longitudinal reinforcement. In the cases examined, this decrease was greater proportionately, as well as numerically. For columns 47 and 48 the deformations after 20 weeks of test indicated that a tension of 90 lb. per sq. in. was present in the concrete, that is, apparently the load carried by the reinforcement exceeded the total load on the columns.

Fig. 14 and Table 6 show that the increase in the stress in the steel was large and that it was greatest for the columns with the smallest percentage of longitudinal reinforcement. The highest

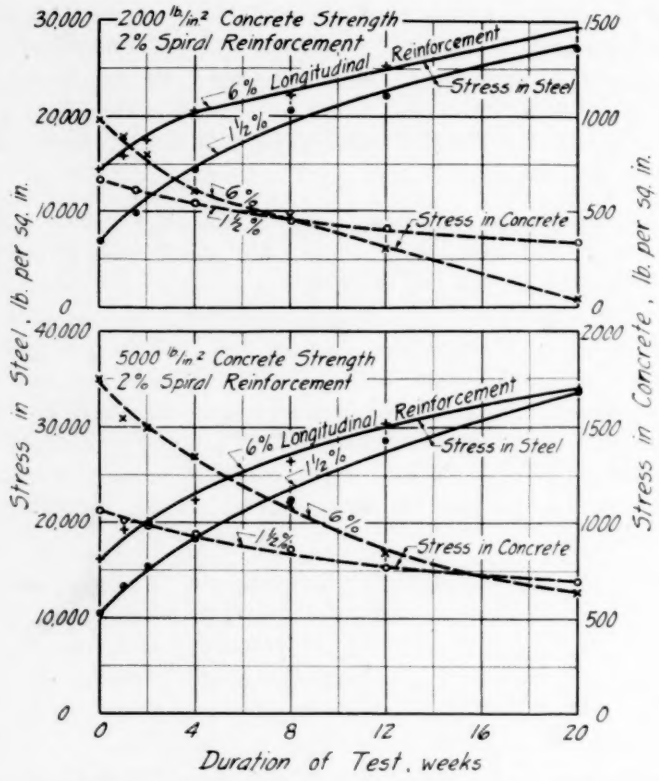


FIG. 14—EFFECT OF DURATION OF TEST ON STRESSES IN STEEL AND CONCRETE

stress reported in this table, 42,660 lb. per sq. in., is practically four times the initial stress. This stress is the average for columns 45 and 46 which have 1½ per cent longitudinal reinforcement. This shows that the steel stress after 20 weeks under the working load was not far from its yield-point stress of 49,500 lb. per sq. in.

For the columns not loaded the shrinkage caused a compression in the steel which resulted in a corresponding tension in the concrete, in spite of the fact that the concrete shortened. The total compressive force found as the product of the measured strain, the modulus of elasticity, and the total area of the steel, must have produced an equal total tensile force in the concrete, since

no external load was applied to the columns. The tensile stresses in the concrete, determined from this relation, are given in Table 6. It will be noted that the average tensile stress in the concrete of columns 107 and 108 was 450 lb. per sq. in. The next lower tensile stress was 370 lb. per sq. in., the average for columns 99 and 100. All four of these columns had 6 per cent longitudinal reinforcement.

A careful examination of all the columns which were stored under no load has recently been made and no cracks have been found. Especial attention was given to the search for cracks on those columns showing the highest tensile stresses. There is no probability that cracks were missed in this visual examination, for their presence would have been plainly indicated in the results of the strain gage measurements which included the total length of the column except for five inches at each end.

18. *Computation of Shrinkage and Flow.*—W. H. Glanville of the Department of Industrial Research, England,[5] has carried out researches on shrinkage and flow in concrete in which tests have been made and theoretical treatments given. The analysis of the problems is so fundamental, and the agreements between the results of the tests and the theoretical treatments are so marked as to make it worth while to include the lines along which the theoretical analysis was carried out. With minor modifications for notations and form of statement, the following are his theoretical analyses of the shrinkage and the flow of reinforced concrete members.

SHRINKAGE

If no slip occurs between the concrete and the steel, the actual change in length of the concrete in a reinforced concrete member is less than the shrinkage of the plain concrete member by an amount equal to the elastic tensile strain of the concrete due to the compressive stress induced in the steel plus the flow of the concrete which has taken place under its tensile stress. Since the increase in load in the steel has to be equal to the decrease in load in the concrete the following equation holds:

$$\Delta f_s \cdot a_s = \Delta f_c \cdot a_c$$

[5]Building Research Technical Papers "Studies in Reinforced Concrete," Technical Paper No. 11, "Shrinkage Stresses," and No. 12, "The Creep or Flow of Concrete Under Load."

where Δf_s and Δf_c are changes in stress in steel and concrete respectively, and a_s and a_c are the areas of steel and concrete respectively.

If c is the flow per unit stress up to the time under consideration the flow occurring during an increment of time Δt is $f_c \cdot \Delta c$, and, since the shrinkage of plain concrete less the flow must be equal to the sum of the strains in the steel and in the concrete, the following equation is derived:

$$\Delta s - f_c \cdot \Delta c = \frac{\Delta f_s}{E_s} + \frac{\Delta f_c}{E_c}$$

$$\text{Since } \Delta f_s \cdot a_s = \Delta f_c \cdot a_c$$

$$\Delta f_s = \frac{a_c}{a_s} \cdot \Delta f_c$$

$$\text{and } \Delta s - f_c \cdot \Delta c = \frac{\Delta f_c}{E_s} \cdot \frac{a_c}{a_s} + \frac{\Delta f_c}{E_c} = \Delta f_c \left(\frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c} \right)$$

$$\text{or writing } \alpha = \frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c}$$

we have in the limit:

$$ds - f_c \cdot dc = \alpha \cdot df_c$$

This relation can be solved if the relations between shrinkage and time and between flow and time are known. If the flow is neglected in the relation between the shrinkage and induced concrete stresses the formula becomes, by integration,

$$s = f_c \left(\frac{1}{E_s} \cdot \frac{a_c}{a_s} + \frac{1}{E_c} \right) = f_s \left(\frac{1}{E_c} \cdot \frac{a_s}{a_c} + \frac{1}{E_s} \right)$$

Mr. Glanville suggests that this formula be used but that the value of E_c be replaced by fictitious or "effective" modulus of elasticity E'_c in such a manner that the effect of the flow is allowed for.

FLOW

If a reinforced concrete member is under a constant load, the flow of the concrete during a small increment of time, Δt, under a unit stress is Δc. The modulus of elasticity of the concrete is assumed to be constant. Since the total flow is equal to the sum of the resulting strains in the concrete and that in the steel the following equation is derived:

$$f_c \cdot \Delta c = \frac{\Delta f_s}{E_s} - \frac{\Delta f_c}{E_c}$$

The increase in load in the steel must be equal to the decrease in the load in the concrete:

$$\Delta f_s \cdot a_s = -\Delta f_c \cdot a_c, \text{ or } \Delta f_s = -\frac{a_c}{a_s} \cdot \Delta f_c$$

substituting this value for Δf_s in the previous equation gives:

$$f_c \cdot \Delta c = -\frac{a_c}{a_s} \cdot \frac{\Delta f_c}{E_s} - \frac{\Delta f_c}{E_c} = -\Delta f_c \left(\frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}\right)$$

$$\text{or} \quad \Delta f_c = -\frac{f_c \cdot \Delta c}{\frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}}$$

$$\text{As before} \quad \alpha = \frac{a_c}{a_s} \cdot \frac{1}{E_s} + \frac{1}{E_c}$$

$$\text{Then} \quad \Delta f_c = -\frac{f_c \cdot \Delta c}{\alpha} \quad \text{or} \quad \frac{\Delta f_c}{f_c} = -\frac{\Delta c}{\alpha}$$

In the limit:

$$\frac{df_c}{f_c} = -\frac{d_c}{\alpha}$$

When $c = 0, f_c = f_o$ where f_o = stress in concrete immediately after loading. This expression may be integrated between the limits f_o and f_c and the result is:

$$f_c = \frac{f_o}{e^{c/\alpha}}$$

where c is the flow of plain concrete under unit stress from the time of loading to the time under consideration, and e is the base of Naperian logarithms.

The stresses in the steel and in the concrete are found from the following relations:

$$f_s = f_a + f'_s$$

$$f'_c = f_o - f_c = f_o \left(1 - \frac{1}{e^{c/\alpha}}\right)$$

$$\text{and} \quad f'_s = f_a \cdot \frac{a_c}{a_s} \cdot \frac{1}{n} \cdot \left(1 - \frac{1}{e^{c/a}}\right)$$

where:

f_s and f_c are the actual stresses in steel and concrete at the time under consideration (disregarding shrinkage effect), f_a and f_o are the stresses in steel and concrete at the time of application of load, f'_s and f'_c are the changes in stress in steel and concrete due to flow, and n is the ratio, $\frac{E_s}{E_c}$, between moduli.

As shrinkage and flow act independently of each other, the combined result is obtained by adding them.

19. *Review of the Problem.*—The findings reported herewith confirm in a striking manner the results of studies made as long ago as 1916. In that year A. H. Fuller and C. C. More presented a paper dealing with "Time Tests of Concrete." At the same meeting E. B. Smith of the Bureau of Public Roads gave a paper on "The Flow of Concrete Under Sustained Loads." Both papers presented evidence of the flow of concrete under load but did not show the strains in reinforcement embedded in the concrete. F. R. McMillan, however, discussed both papers and described results of measurements which he had initiated on reinforced concrete columns in buildings in service. The observations were begun about two months after the placing of the columns and had been in progress about seven months at the time of the first report. It was already evident that considerable shortening of the concrete and steel was taking place. In the following year (1917) Smith presented to the American Concrete Institute a paper embodying the results of a continuation of his studies. McMillan discussed this paper also, showing that the strains in the columns under measurement had reached a value corresponding, in one instance (designated as column 19), to a stress in the steel as high as 36,000 lb. per sq. in. In 1921 M. B. Lagaard presented as discussion of McMillan's paper "A Study of Column Test Data" further results of the studies initiated by McMillan on columns in service. By that time the measurements had been carried on for approximately six years. The strains had increased so that the apparent stress in the steel of column 19 was 45,000 lb. per sq. in. It is not unlikely that even

then the yield-point stress had been exceeded, especially in view of the fact that the measurements were not started until the columns were two months old. As no comment was made on the appearance of the column it is probable that there were no indications of distress.

R. E. Davis reports that in his investigation of flow the strength of plain concrete was uninfluenced by having been subjected to a long-time load of 800 lb. per sq. in. which caused a final deformation of 900 millionths or three times that with which the test started. Other of his tests confirm these results. If, therefore, the flow in a reinforced concrete column continues to the point at which the steel is stressed beyond the yield point, it is reasonable to expect that the concrete is still capable of carrying its full share of the load.

The only occasion for concern then, must be as to whether the carrying capacity of the steel is reduced by being stressed beyond the yield point. The tests have not yet been carried far enough to give the answer from the experimental end, but the question may reasonably be asked: If the longitudinal steel receives from the spiral and from its embedment in the concrete, sufficient lateral support to prevent buckling under the yield-point stress, how can it escape from continuing to carry its full share of the load?

At the risk of anticipating too far what the final outcome will be, it seems imperative that enough speculation be indulged in to enable us to visualize what must yet be done before the reinforced concrete column problem is solved. First, the strength of the column appears to depend upon the sum of the strengths of the concrete and the longitudinal reinforcement regardless of the ratio of the moduli of elasticity. Second, it seems evident that the basis for the design of reinforced concrete columns must be changed accordingly.

The tests must be carried far enough to determine what is the highest axial load that a reinforced concrete column will carry indefinitely without failure.

Tests must also be carried out on columns subjected to combined bending and direct stress. These tests must be made in such a way that some generalization may be made as to what

redistribution of the bending moment initially present in an indeterminate structure takes place as the concrete shrinks, and flows under long time load.

Readers are referred to the JOURNAL *for June, 1931, for discussion of the Reinforced Concrete Column Investigation of Committee 105. Such discussion should reach the Secretary by May 1, 1931.*

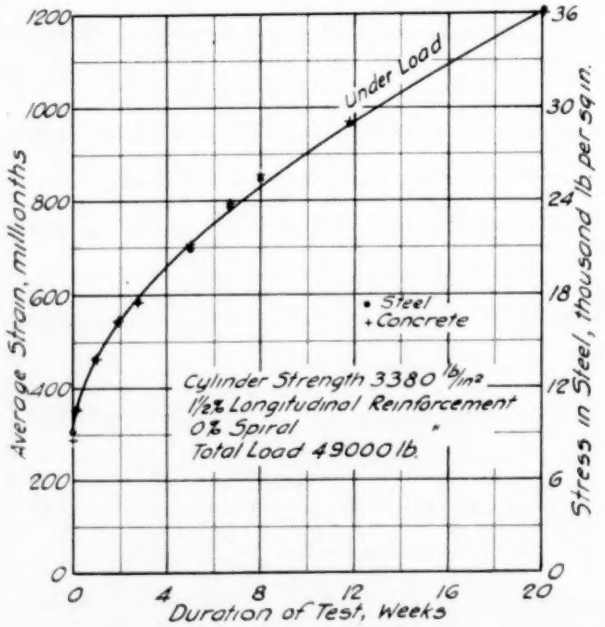


FIG. 15—DEFORMATION OF COLUMNS 1 AND 2 UNDER SUSTAINED LOAD IN DRY STORAGE

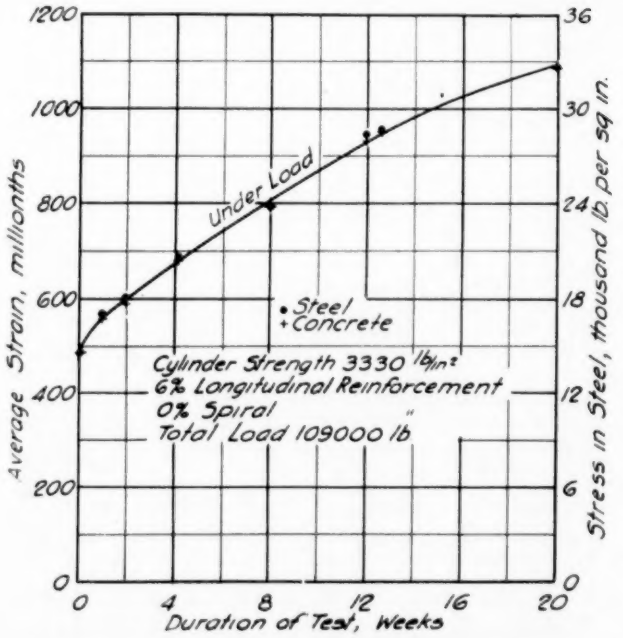


FIG. 16—DEFORMATION OF COLUMNS 3 AND 4 UNDER SUSTAINED LOAD IN DRY STORAGE

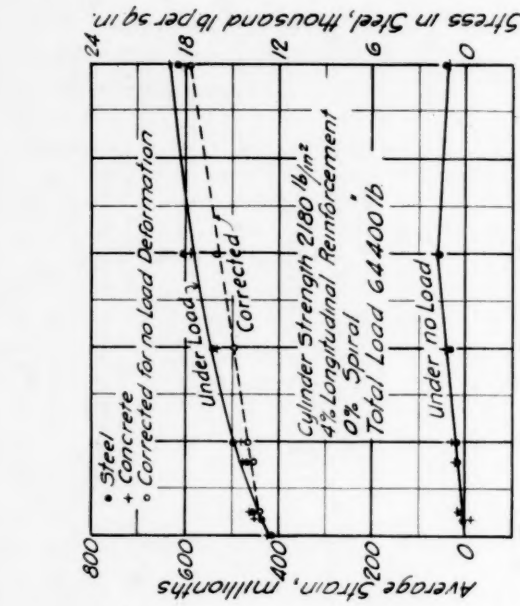


FIG. 18—DEFORMATION OF COLUMNS 9 AND 10 UNDER SUSTAINED LOAD AND OF COLUMNS 11 AND 12 UNDER NO LOAD IN WET STORAGE

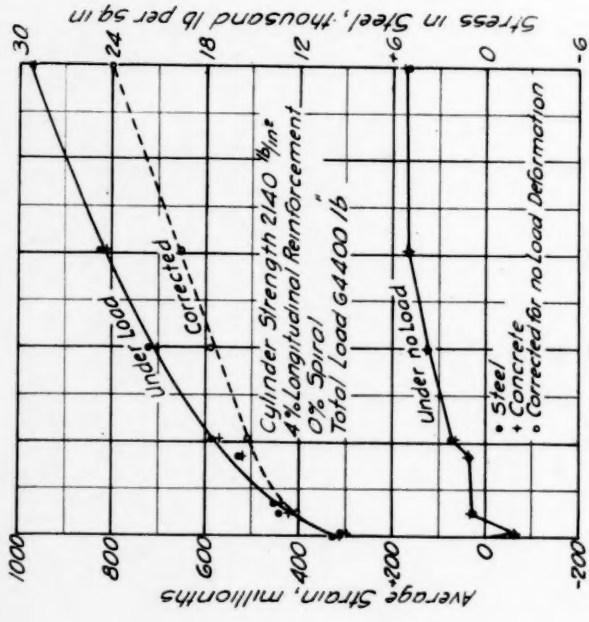


FIG. 17—DEFORMATION OF COLUMNS 5 AND 6 UNDER SUSTAINED LOAD AND OF COLUMNS 7 AND 8 UNDER NO LOAD IN DRY STORAGE

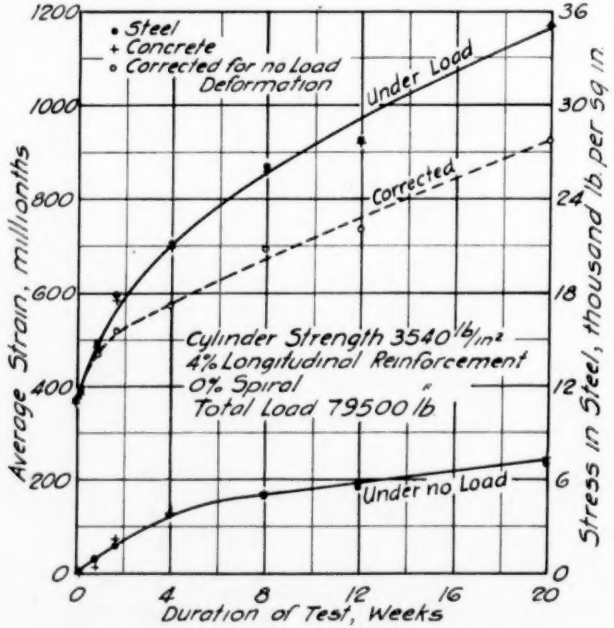


FIG. 19 (LEFT)—DEFORMATION OF COLUMNS 13 AND 14 UNDER SUSTAINED LOAD AND OF COLUMNS 15 AND 16 UNDER NO LOAD IN DRY STORAGE

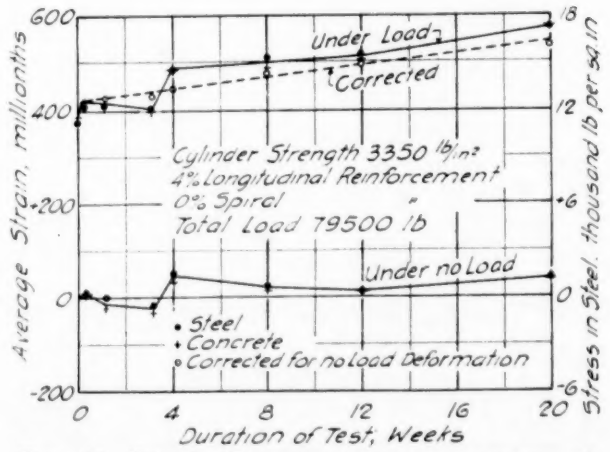


FIG. 20 (ABOVE)—DEFORMATION OF COLUMNS 17 AND 18 UNDER SUSTAINED LOAD AND OF COLUMNS 19 AND 20 UNDER NO LOAD IN WET STORAGE

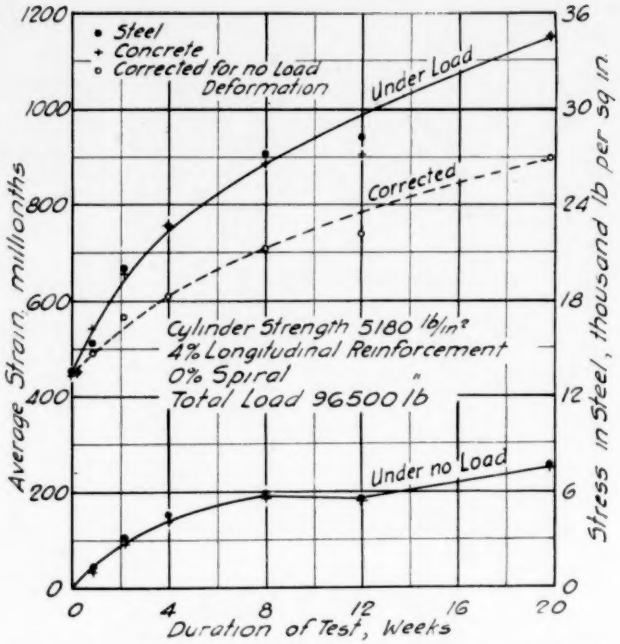


FIG. 21—DEFORMATION OF COLUMNS 21 AND 22 UNDER SUSTAINED LOAD AND OF COLUMNS 23 AND 24 UNDER NO LOAD IN DRY STORAGE

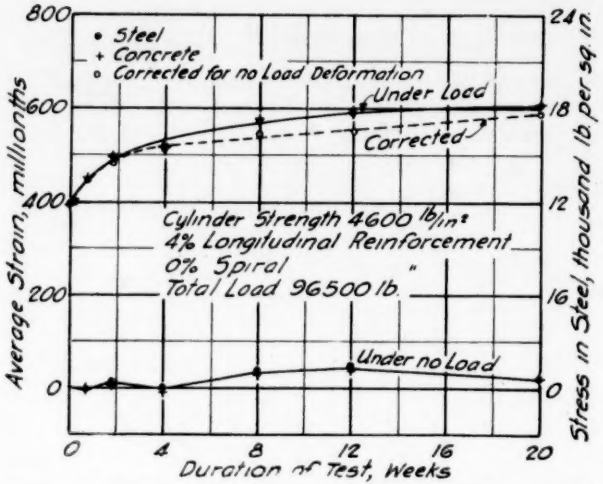


FIG. 22—DEFORMATION OF COLUMNS 25 AND 26 UNDER SUSTAINED LOAD AND OF COLUMNS 27 AND 28 UNDER NO LOAD IN WET STORAGE

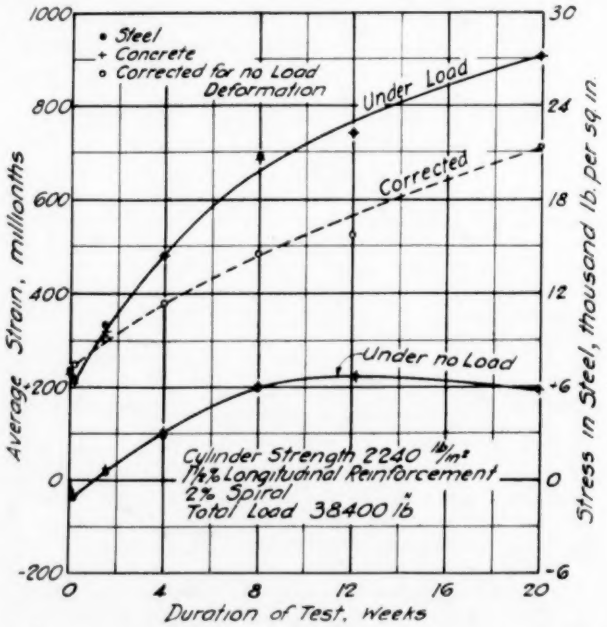


FIG. 23—DEFORMATION OF COLUMNS 29 AND 30 UNDER SUSTAINED LOAD AND OF COLUMNS 31 AND 32 UNDER NO LOAD IN DRY STORAGE

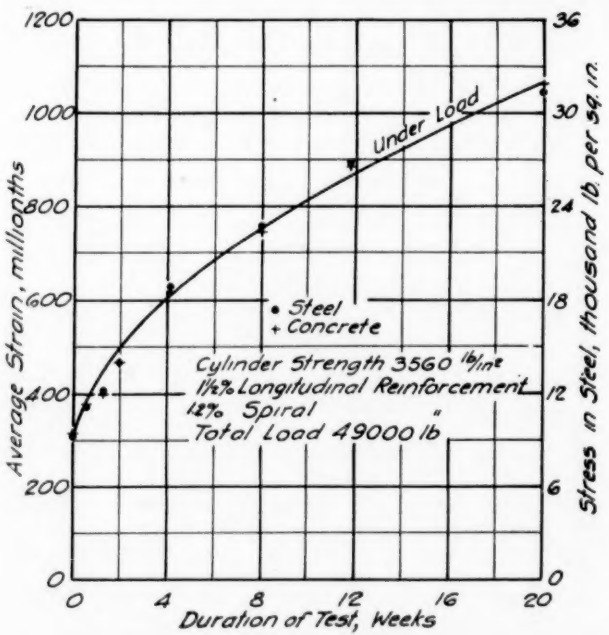


FIG. 24—DEFORMATION OF COLUMNS 33 AND 34 UNDER SUSTAINED LOAD IN DRY STORAGE

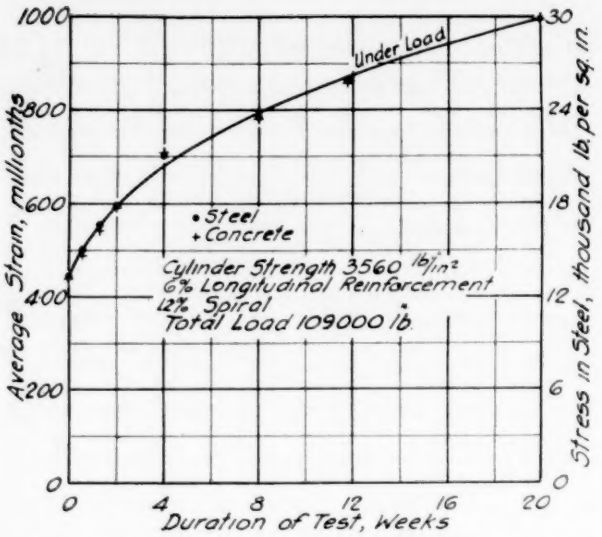


FIG. 25 (ABOVE)—DEFORMATION OF COLUMNS 35 AND 36 UNDER SUSTAINED LOAD IN DRY STORAGE

FIG. 27 (BELOW)—DEFORMATION OF COLUMNS 41 AND 42 UNDER SUSTAINED LOAD AND OF COLUMNS 43 AND 44 UNDER NO LOAD IN DRY STORAGE

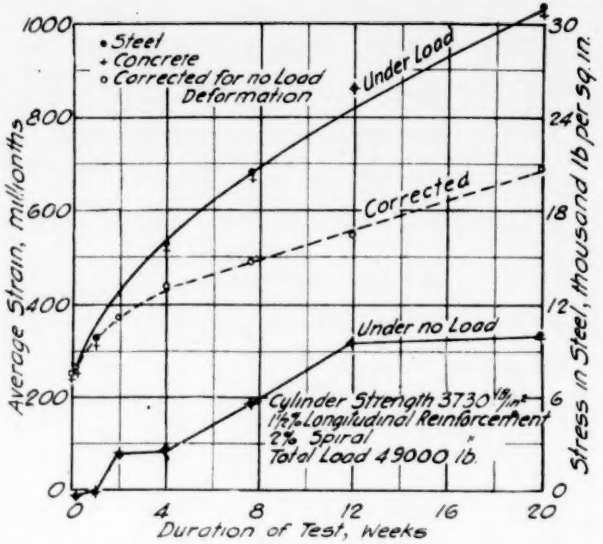
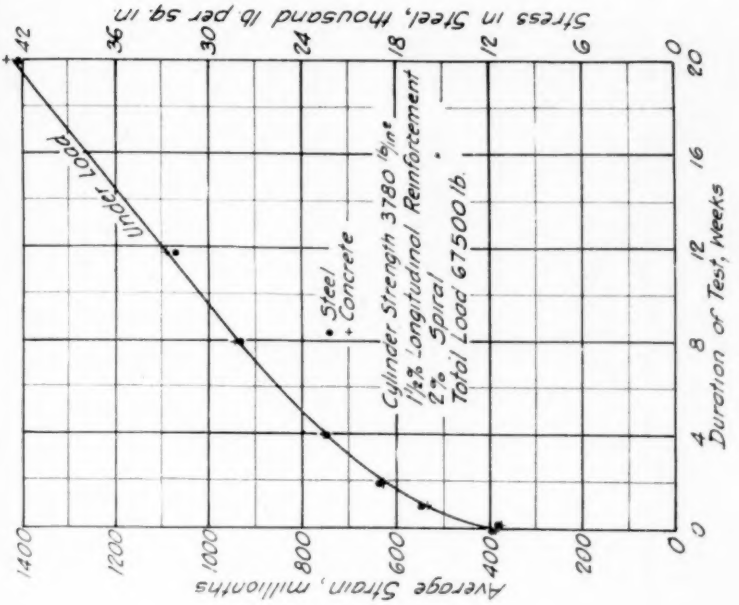
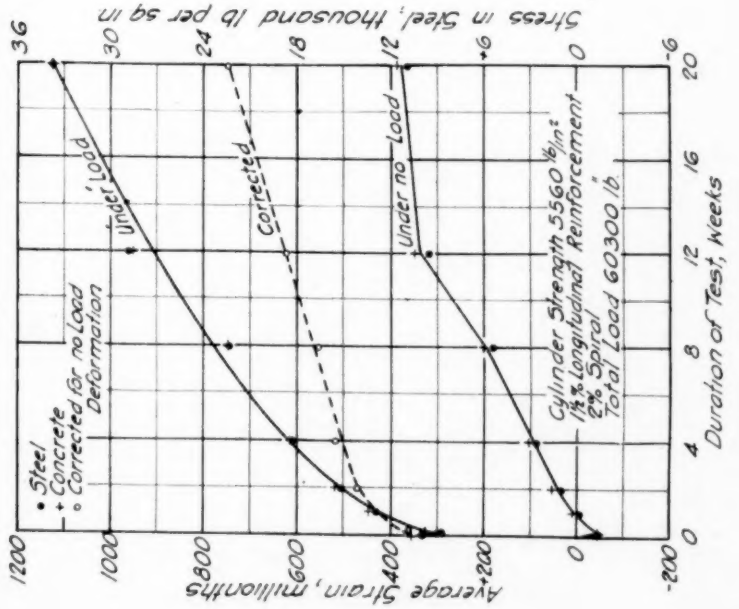


FIG. 26 (ABOVE)—DEFORMATION OF COLUMNS 37 AND 38 UNDER SUSTAINED LOAD AND OF COLUMNS 39 AND 40 UNDER NO LOAD IN DRY STORAGE

FIG. 28 (BELOW)—DEFORMATION OF COLUMNS 45 AND 46 UNDER SUSTAINED LOAD IN DRY STORAGE


Stress in Steel, thousand lb per sq in
Average Strain, millionths
Duration of Test, Weeks
• Steel
+ Concrete
Under Load
Cylinder Strength 3780 lb/in²
1¼% Longitudinal Reinforcement
2% Spiral
Total Load 67500 lb.


Stress in Steel, thousand lb per sq in
Average Strain, millionths
Duration of Test, Weeks
• Steel
+ Concrete
○ Corrected for no Load Deformation
Under Load
Corrected
Under no Load
Cylinder Strength 5560 lb/in²
1¼% Longitudinal Reinforcement
2% Spiral
Total Load 60300 lb.

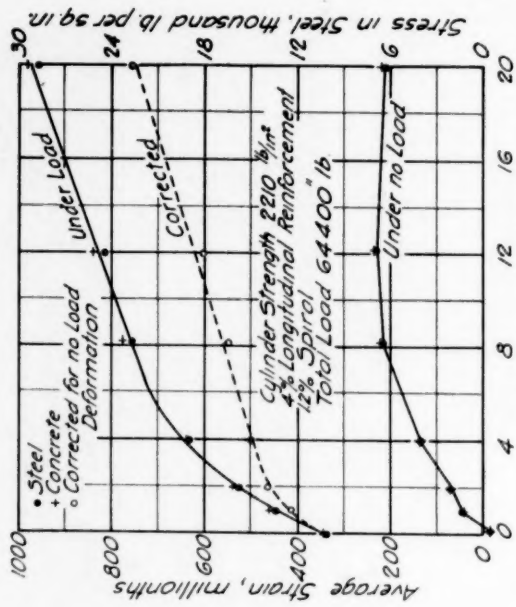


FIG. 30—DEFORMATION OF COLUMNS 49 AND 50 UNDER SUSTAINED LOAD AND OF COLUMNS 51 AND 52 UNDER NO LOAD IN DRY STORAGE

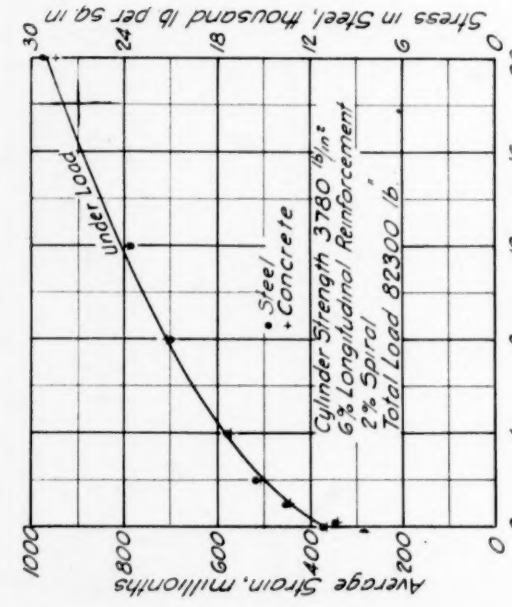


FIG. 29—DEFORMATION OF COLUMNS 47 AND 48 UNDER SUSTAINED LOAD IN DRY STORAGE

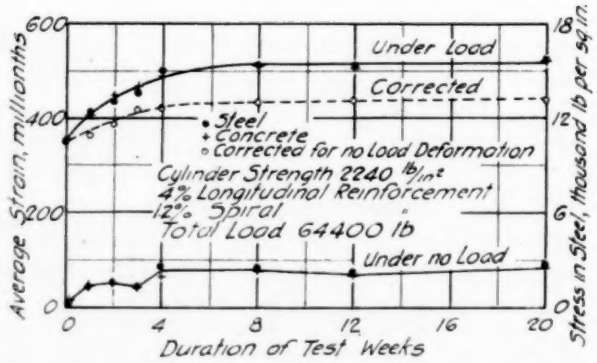


FIG. 31—DEFORMATION OF COLUMNS 53 AND 54 UNDER SUSTAINED LOAD AND OF COLUMNS 55 AND 56 UNDER NO LOAD IN WET STORAGE

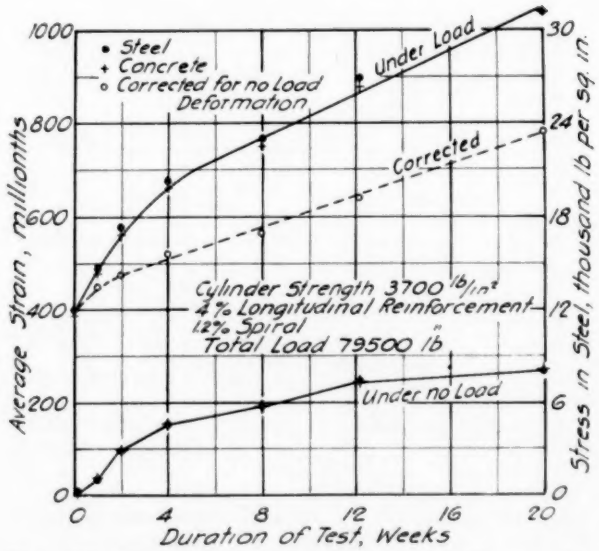


FIG. 32—DEFORMATION OF COLUMNS 57 AND 58 UNDER SUSTAINED LOAD AND OF COLUMNS 59 AND 60 UNDER NO LOAD IN DRY STORAGE

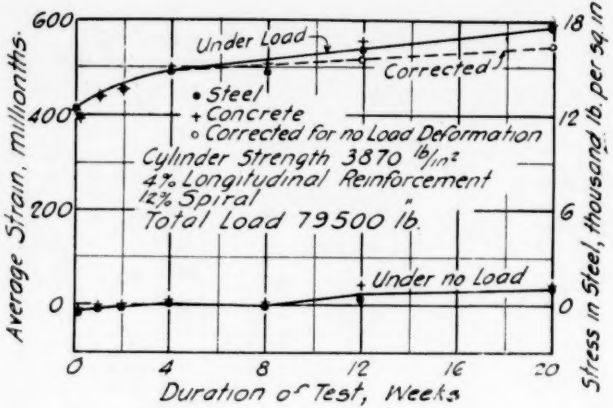


FIG. 33—DEFORMATION OF COLUMNS 61 AND 62 UNDER SUSTAINED LOAD AND OF COLUMNS 63 AND 64 UNDER NO LOAD IN WET STORAGE

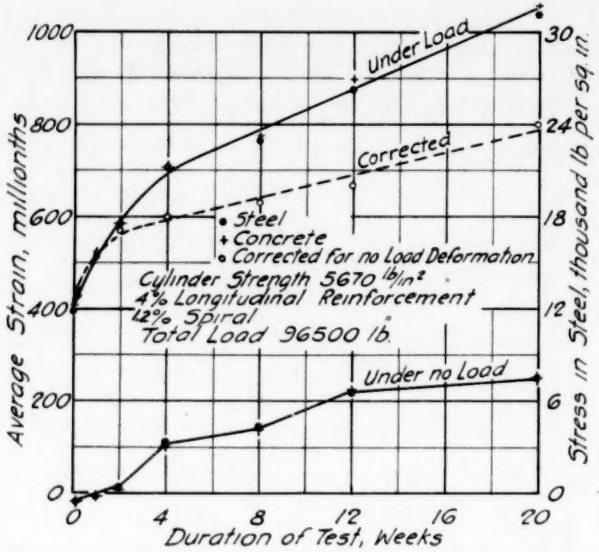


FIG. 34—DEFORMATION OF COLUMNS 65 AND 66 UNDER SUSTAINED LOAD AND OF COLUMNS 67 AND 68 UNDER NO LOAD IN DRY STORAGE

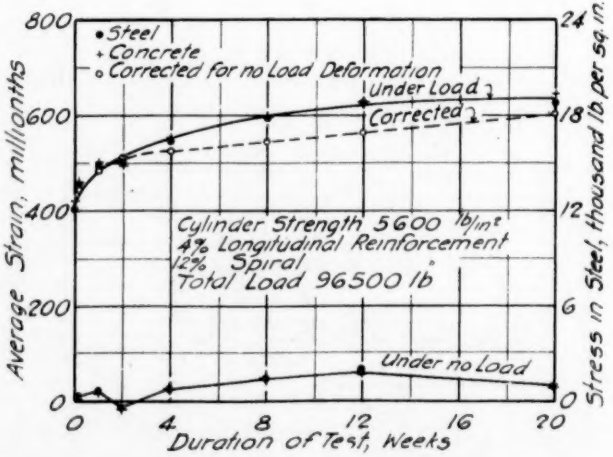


FIG. 35—DEFORMATION OF COLUMNS 69 AND 70 UNDER SUSTAINED LOAD AND OF COLUMNS 71 AND 72 UNDER NO LOAD IN WET STORAGE

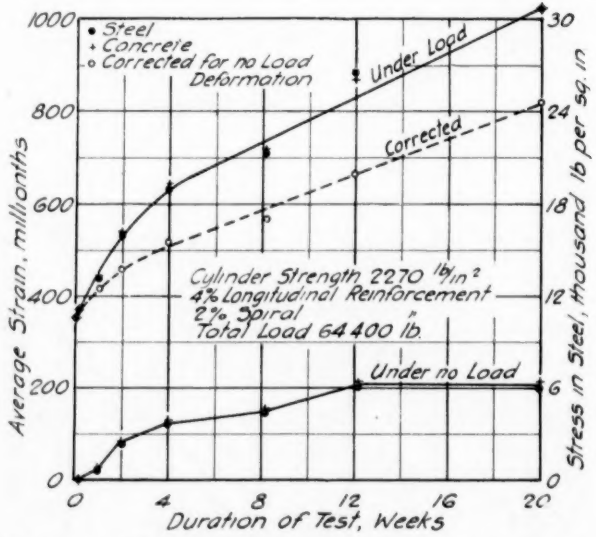


FIG. 36—DEFORMATION OF COLUMNS 73 AND 74 UNDER SUSTAINED LOAD AND OF COLUMNS 75 AND 76 UNDER NO LOAD IN DRY STORAGE

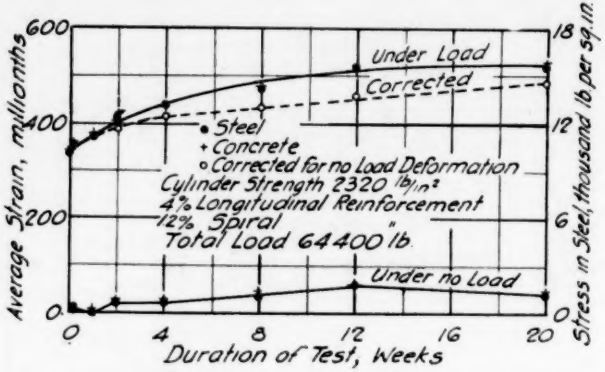


FIG. 37—DEFORMATION OF COLUMNS 77 AND 78 UNDER SUSTAINED LOAD AND OF COLUMNS 79 AND 80 UNDER NO LOAD IN WET STORAGE

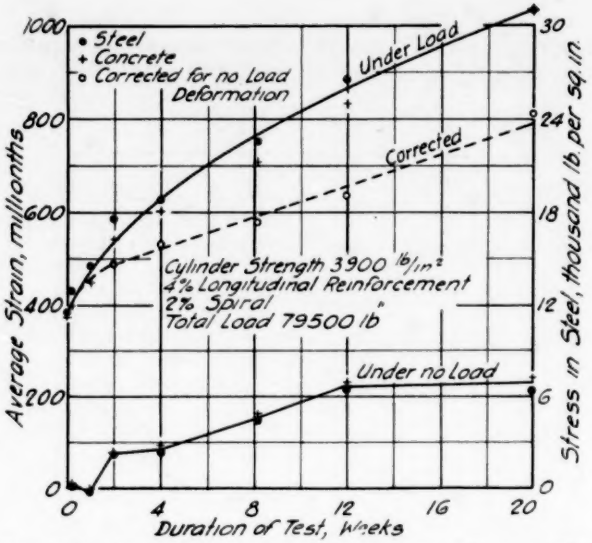


FIG. 38—DEFORMATION OF COLUMNS 81 AND 82 UNDER SUSTAINED LOAD AND OF COLUMNS 83 AND 84 UNDER NO LOAD IN DRY STORAGE

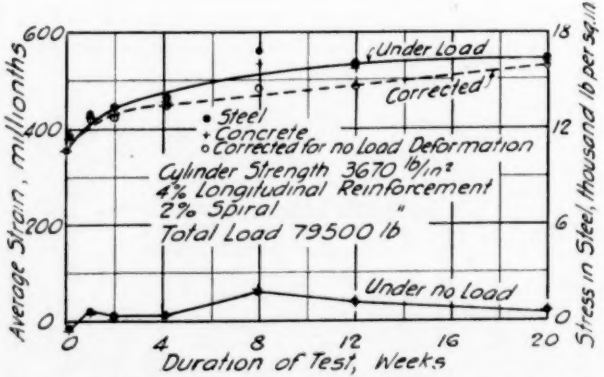


FIG. 39—DEFORMATION OF COLUMNS 85 AND 86 UNDER SUSTAINED LOAD AND OF COLUMNS 87 AND 88 UNDER NO LOAD IN WET STORAGE

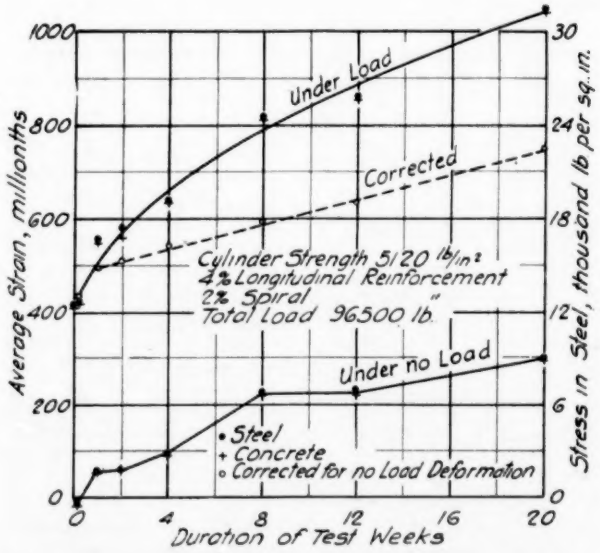


FIG. 40—DEFORMATION OF COLUMNS 89 AND 90 UNDER SUSTAINED LOAD AND OF COLUMNS 91 AND 92 UNDER NO LOAD IN DRY STORAGE

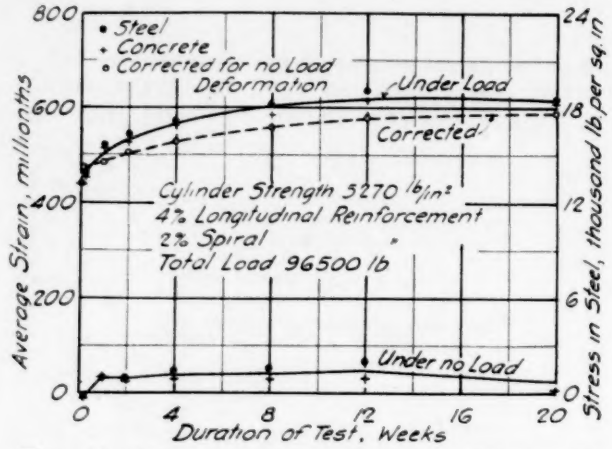


FIG. 41—DEFORMATION OF COLUMNS 93 AND 94 UNDER SUSTAINED LOAD AND OF COLUMNS 95 AND 96 UNDER NO LOAD IN WET STORAGE

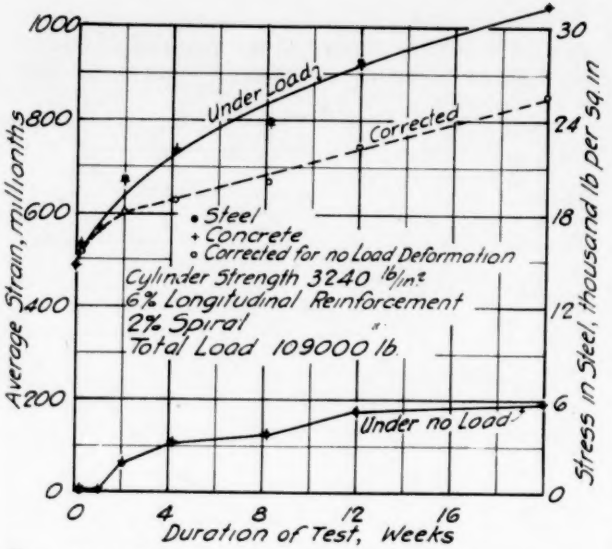


FIG. 42—DEFORMATION OF COLUMNS 97 AND 98 UNDER SUSTAINED LOAD AND OF COLUMNS 99 AND 100 UNDER NO LOAD IN DRY STORAGE

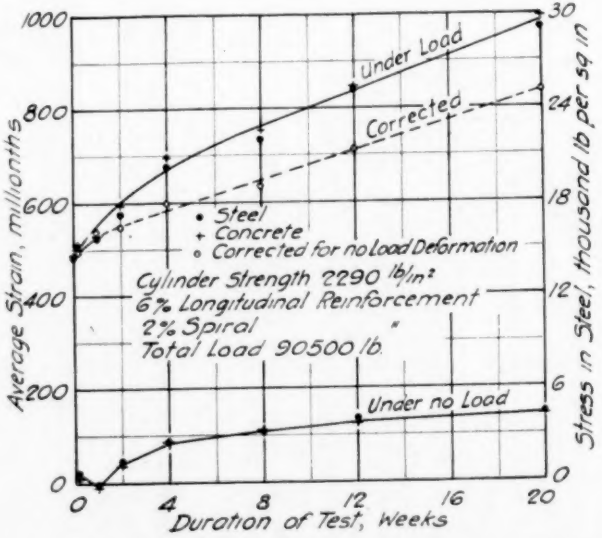


FIG. 43—DEFORMATION OF COLUMNS 101 AND 102 UNDER SUSTAINED LOAD AND OF COLUMNS 103 AND 104 UNDER NO LOAD IN DRY STORAGE

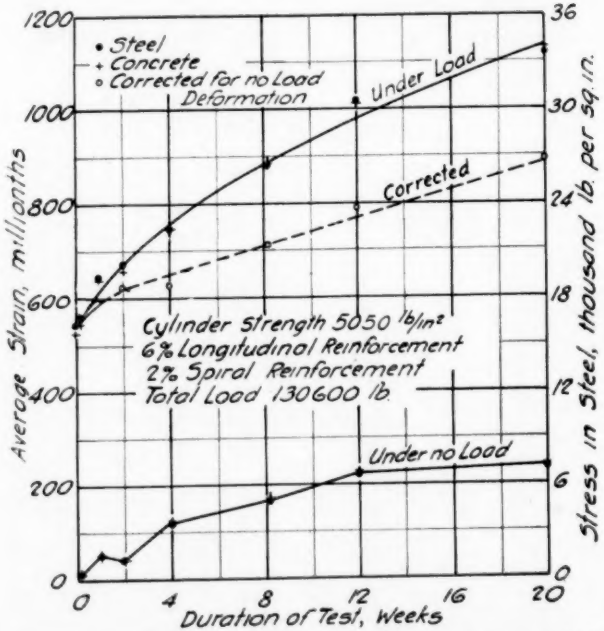


FIG. 44—DEFORMATION OF COLUMNS 105 AND 106 UNDER SUSTAINED LOAD AND OF COLUMNS 107 AND 108 UNDER NO LOAD IN DRY STORAGE

Flow of Concrete under the Action of Sustained Loads

BY RAYMOND E. DAVIS* AND HARMER E. DAVIS†

This paper is here published to encourage discussion at the 27th annual convention at which it will be presented formally in abstract. It represents a part of the work assigned to Committee 109—Summary of Plastic Flow. Professor Davis is Author-Chairman and Harmer E. Davis is a member of the committee. It is solely their joint effort in which the committee as a whole has not participated.—EDITOR

SYNOPSIS

THIS PAPER presents the results of tests on plain and reinforced concrete to determine the effect of loads sustained over a considerable period of time upon the deformations and upon the distribution of stresses in concrete and steel.

Part 1 of the paper describes laboratory tests at the University of California in the interval 1925-1930 on more than 200 cylinder specimens, all maintained under controlled conditions of storage as regards temperature and humidity, to determine the influence of a constant sustained compressive stress upon axial deformations. The deformation due to stress which takes place immediately upon the application of the load is here called the "instantaneous deformation," and that which takes place subsequent to the application of load is here designated as the "flow," a quantity separate from changes in dimensions due to such causes as variations in temperature and moisture conditions. Likewise the ratio of unit stress to unit instantaneous deformation is termed the "modulus of elasticity," and the ratio of the sustained unit stress to the total deformation due to load (instantaneous plus flow), is called the "sustained modulus of resistance."

The tests were made to determine the manner in which flow is influenced by such elements as (1) length of period of sustained load, (2) magnitude of sustained load, (3) age at the time of loading, (4) character of mineral aggregate, (5)

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richness of mix, (6) gradation of aggregate, (7) water-cement ratio, (8) reinforcement, (9) alternately applying and releasing load, and (10) moisture conditions of storage, including variations in humidity of surrounding air.

From these tests, it has been determined among other things that the flow of plain concrete under a constantly sustained compressive stress (1) continues to increase, at first rapidly and then more slowly, for a long time, certainly for several years, (2) is greater for concrete in an atmosphere of low humidity than for an identical concrete in an atmosphere of high humidity or when fully saturated with water, (3) is materially affected by the character of the aggregate, being substantially larger for sandstone concrete than for limestone concrete, (4) is less for rich mixes than for lean ones, (5) is less for concrete having a high modulus of elasticity than for one having a low modulus of elasticity, (6) is greater for a high water-cement ratio than for a low one, and (7) is greater when load is applied at an early age than when it is applied at a later age.

Longitudinal reinforcement in specimens stored in air appreciably reduces both the flow and shrinkage, but under the action of a load long sustained, the compressive stress in the longitudinal steel continues to increase until it becomes several times the stress which existed immediately upon application of the load, even reaching the yield point of the steel.

When after a time the load is released, there is an elastic recovery which is immediate, and a plastic recovery which increases with time, but the plastic recovery is less than the plastic flow which took place under the sustained loading. When concrete has been so loaded and unloaded, the instantaneous modulus of elasticity becomes somewhat greater than that of a concrete alike in all other respects save that it has not been previously stressed.

In time, even at normal working stresses, the flow of concrete may be several times as great as the initial instantaneous deformation, or, putting it in another way, the instantaneous modulus of elasticity is several times the sustained modulus of resistance.

When the load is alternately applied and released for a given time, the magnitude of the instantaneous deformation and the rate of plastic flow and plastic recovery tend to become less as the number of stress cycles is increased.

Part 2 of the paper summarizes the published results of the more important tests reported by other investigators during the last 25 years, in which the phenomenon of "time yield," "plasticity," "time deformation," "plastic flow" or "flow" has been observed and discussed. In general, while the conditions surrounding these early tests are not sufficiently well known nor the tests sufficiently extensive to make the quantitative data of direct value, the results consistently point to a marked time effect, both upon laboratory specimens and upon service structures.

Tests which are here summarized include those reported by Hatt in 1907 on beams; by McMillan in 1915, 1916 and 1921, on beams, slabs, columns; by Fuller and More in 1915, on cylinders and flat slabs; by Goldbeck and Smith in 1914-1917, on cylinders, beams and slabs; by Lord in 1914-16, on flat slabs; and by Faber, 1916-27, on beams.

INTRODUCTION

FOR MORE than a quarter of a century the property possessed by all concrete, to a greater or less degree, of yielding slowly under the action of sustained loads has been known to exist, first through the gradually increasing deflection of beams and slabs and later through measurements of the strains in both concrete and steel of columns, beams and slabs. This property, which is entirely separate from shrinkage due to gradual release of uncombined water when concrete is in air, renders concrete, even within working stresses, not a perfectly elastic material, but a material which when subjected to sustained load continues slowly to yield or deform for a long time. That is, not only does the stress-strain relation of a given concrete vary with the magnitude of the stress, but also with the length of time over which the stress is sustained.

This property of yielding with the passage of time has been designated as "time yield," "time deformation," "plastic deformation," and "plastic flow." In this report, except in quoting the words of other investigators, the term "flow" will be employed, though it is problematical whether there actually occurs in concrete a movement of the particles over one another in the manner commonly associated with the flow of any material.

In many structures where dead load forms a large part of the total load this slow yielding of the concrete with time would seem to bring about a stress distribution quite different from that which would exist under the action of an equivalent live load. In buildings and other structures exposed to drying conditions there also enters further to complicate the problem the added factor of shrinkage of the concrete, which likewise is gradually accumulative over a long period. Take the simple case of a reinforced column, for example; due to dead load, the stress in the concrete may gradually diminish and the stress in the longitudinal steel gradually increase as flow and shrinkage take place, until the yield point of the steel is reached, after which the distribution of stress between steel and concrete may perhaps remain substantially constant. Or consider a simply supported reinforced beam, in which the flexural stress is insufficient to produce cracks below the neutral axis and which is free to contract longitudinally;

due to shrinkage alone the steel is gradually placed in compression, with the top fibers of the beam in compression and the bottom fibers in tension, and due to dead load deformations there may be gradually produced a lowering of the neutral axis tending to increase the tension in the steel and to decrease the compression in the extreme fiber of the concrete. Thus the effect of flow may be to decrease the stress in the concrete and the effect of shrinkage may be to decrease the tensile stress in the steel. In restrained beams, continuous frames, and arches it appears likely that flow acts in some degree to relieve the stresses at sections of large moment and to redistribute the moment in a manner more nearly to equalize negative and positive moments. For example, it might be expected that for a beam fixed at the ends and carrying a uniform dead load, the points of contra flexure would gradually shift position and the elastic curve gradually change its shape until, in the process of time, the maximum positive and negative moments were substantially the same. Or for an arch where there was a gradual rib shortening, due to whatever cause, it appears probable that the moments at crown and spring lines may be substantially less than would be the case were the length of the arch ring abruptly changed by the same amount. And the dead-load moment diagram for a continuous beam with concentrated loads long sustained, instead of being a series of connected straight lines, may be a curved line with summits and depressions rounded.

Many other examples might be cited, but those mentioned are sufficient to indicate the desirability of determining the facts concerning flow and of considering the influence of flow upon structural design. The recognized importance of flow as a factor in stress analysis has prompted the preparation of this paper. Here attempt is made to present quantitative data concerning the phenomenon of flow as determined by tests, leaving to others the task of discussing the significance of flow as related to stresses in structures.

The paper is divided into two parts:

Part 1 describes in considerable detail tests made in the interval 1925-30 at the University of California under the direction of the

authors upon more than 200 compression cylinders under closely controlled conditions. Here the influence of various factors upon flow are discussed.

Part 2 is a summary of some of the more important papers and discussions published in the last 25 years in which the effects of flow, as determined by tests, have been described. These include laboratory tests on cylinders, beams and slabs, and field observations upon the several structural elements. Following the abstracts is a bibliography of articles and papers dealing with flow and shrinkage.

In this paper the term "flow" will be taken to mean that axial deformaton due to stress which takes place subsequent to the application of a sustained load; that is, the flow at the end of any interval of time is the deformation due to the sustained load but not to the instantaneous application of the load. It is the total deformation due to stress, minus the deformation which takes place immediately upon the application of the load. It does not include changes in length due to causes other than stress, such as shrinkage and variations in temperature.

Since concrete under the action of sustained stress is so imperfectly elastic, it seems particularly inappropriate to use the term "modulus of elasticity" as being the ratio of unit stress to unit deformation when flow occurring with the passage of time is included. Consequently in this report the term "modulus of elasticity" will be employed only to designate the ratio of unit stress to unit strain at the time of application of load, and the term "modulus of resistance" will be employed to designate the ratio of unit stress to total unit deformation, this total to include the "instantaneous deformation" occurring immediately upon the application of load, plus the deformation, due to sustained stress, occurring subsequent to the application of load.

Just as concrete instantaneously deforms upon application and continues to deform for a long period if the load is sustained, so upon release of load is there an immediate reduction in deformation, here called the "instantaneous recovery," followed by a further gradual reduction in deformation which extends over a considerable period, here designated as the "plastic recovery."

PART 1—UNIVERSITY OF CALIFORNIA TESTS

Remarks

The tests here reported were conducted in the Materials Testing Laboratory of the University of California to determine the behavior of concrete under long continued compressive stress. From a modest beginning in 1925, the program has been periodically enlarged until in 1930 the number of specimens had reached a total of 217, of which 131 were subjected to sustained stress and 86 were maintained in an unstressed condition, being under observations to determine dimensional changes (shrinkage or expansion) due to causes other than stress. As this paper is written, a considerable number of the specimens have been under observation for more than four years.

All tests have been carried out under closely controlled conditions, the specimens being constantly maintained at 70 deg. F. and the atmospheric humidity being constantly maintained at a desired value. In this manner the uncertainties arising from variable temperature and humidity have been eliminated or the effects of known variations in the humidity have been observed.

For every specimen which has been subjected to sustained stress, there has been a similar specimen maintained under identical conditions except that it has been subjected to no load. These have been designated as "controls," for by means of them it has been possible to determine the *flow* of each loaded specimen by deducting from the total observed deformation, the observed shrinkage or expansion of its *control*.

The specimens have been cylinders varying from 4- to 10- in. diameter and from 12- to 24- in. length. Among the specimens a constant sustained compressive stress as low as 200 lb. per sq. in., and as high as 1200 lb. per sq. in. has been maintained, and the ages at the time of loading have varied from 3 days to 7 months.

Acknowledgments

The tests here described have been made possible through the generous financial support rendered by the Research Board of the University of California, by Engineering Foundation, and by the American Society of Civil Engineers, through its Committee on Arches.

In the years since the tests were started more than a dozen men at one time or another have contributed their services to the work, and to this group thanks are due to the painstaking manner in which the tests have been executed. In particular it is desired to acknowledge the valuable assistance rendered by G. E. Troxell, Associate Professor of Civil Engineering, who supervised many of the earlier tests; J. L. Banville, Assistant in the Materials Testing Laboratory who has been immediately in charge of much of the work since its beginning and who in a large measure is responsible for the design of special instruments and apparatus; and J. S. Hamilton, Assistant in the Materials Testing Laboratory, who has reviewed and assembled many of the data and who has supervised many of the later tests.

Scope of the Tests

The test program has been designed to show the effect upon the time-flow relation of the following:

1. Richness of mix,
2. Gradation of the aggregate,
3. Mineral composition of the aggregate,
4. Age at time of applying load,
5. Condition of storage as regards moisture,
6. Magnitude of stress,
7. Reinforcement,
8. Alternately applying and releasing loads.

Also, tests have been made to determine the effect of flow upon the strength and elasticity.

The tests are divided into seven series, as follows:

Series 1—Inaugurated in 1925 to determine the effect of richness of mix and gradation of the aggregate upon flow. Completed in 1926.

Series 2—Inaugurated in 1926 to determine the effect of magnitude of unit stress, age at time of loading and moisture conditions upon flow. Tests still in progress.

Series 3—Inaugurated in 1928 to determine the effect of humidity upon flow. Still in progress.

Series 4—Inaugurated in 1928 to determine the effect of character of the mineral aggregate upon flow. Still in progress.

Series 5—Inaugurated in 1929 to determine the effect of reinforcement upon flow and flow upon stress in reinforcement. Still in progress.

Series 6—Inaugurated in 1929 to study the effect of alternately applying and releasing a constant sustained load upon flow. Still in progress.

Series 7—Inaugurated in 1929 to study the effect of richness of mix and moisture conditions upon flow and to determine the effect of flow upon the modulus of elasticity and compressive strength. Completed December 1930.

Further details concerning the several series follow:

Series 1

The purpose of these tests was to determine the effect upon flow of (1) richness of mix, (2) gradation of aggregate, and (3) water-cement ratio.

The specimens were 6 by 24-in. cylinders, 16 in number, 12 of which were subjected to a continuous compressive stress of 640 lb. per sq. in. and 4 of which were maintained in an unstressed condition. All specimens were cured for 2 months in water and 5 months in air prior to the application of load. After the beginning of the loading tests, all specimens, including those of the unstressed group, were stored in an atmosphere of 70 deg. F. and 78 per cent relative humidity.

Four concrete mixes were investigated, differing from one another either in regard to richness or to gradation of aggregate, all as indicated in Table 1.

TABLE 1—PROPORTIONS—SERIES 1

Mix	Cement-Aggregate Ratio		Fineness Modulus of Aggregate	Water-Cement Ratio by Volume
	By Volume	By Weight		
1	1:3.4	1:4	6.43	0.81
2	1:3.4	1:4	3.62	1.06
3	1:5.9	1:7	6.43	1.01
4	1:5.9	1:7	3.62	1.42

For mixes of low fineness modulus (mixes 2 and 4) the aggregate was composed of Niles top gravel, a Coast Range sand all of which passed a No. 4 sieve. For mixes of high fineness modulus the fine aggregate was Niles top gravel and the coarse aggregate was a crushed metamorphic sandstone between the sizes ½ and 1½ in. from the local quarries of the Oakland Paving Co. From each mix there were manufactured four specimens, three of which were loaded and one of which maintained without stress.

Series 2

The purpose of these tests was to determine the effect of (1) magnitude of sustained stress, (2) age at time of loading, and (3) air *versus* water storage.

The specimens were 4 by 14-in. cylinders, 58 in number, 40 of which were loaded and 18 of which were unstressed.

The concrete was the same for all specimens, being composed of 1 part cement to 5.05 parts, by weight, of an aggregate composed entirely of crushed granite for which the fineness modulus was 5.03. The proportioning and materials were the same as those employed in the concrete used in the construction of the Stevenson Creek Test Dam of the Arch Dam Investigation of Engineering

Foundation. The water-cement ratio was 1.03 by volume and the average slump was 3.2 in. The mix data are given in Table 2.

TABLE 2—PROPORTIONS—SERIES 2

	Cement	Fine Aggregate	Medium Aggregate	Coarse Aggregate
Proportions by Weight	1	3.01	1.15	0.89
Sieve sizes		Below ⅜ in.	⅜ to ¾ in.	¾ to 1½ in.
Fineness Modulus		3.27	6.72	8.78

In Table 3 are given the storage and stress conditions investigated. Examining the table it will be found that the age at time of loading varied from 2 days to 3 months and that the intensity of sustained stress varied from 200 to 1200 lb. per sq. in. In general, observations were made upon 3 control specimens for each age of loading and storage conditions, and observations were made upon 3 loaded specimens for each age of loading. storage condition and intensity of sustained stress. For those specimens stored in water, the

TABLE 3—STORAGE AND STRESS CONDITIONS OF SPECIMENS—SERIES 2

Curing	Storage After Loading	Age at Loading	Stress, Lb. Per Sq. In.
Water	Water	2 days	200
Water	Water	7 days	300 600
Damp Sand	Water	28 days	300 600 900
Damp Sand	Water	3 months	600 900 1200
Damp Sand	Air at 70 per cent relative humidity	28 days	300 600 900
Damp Sand	Air at 70 per cent relative humidity	3 months	600 900 1200

temperature of the bath was 70 deg. F., and for those stored in air the temperature of the surrounding atmosphere was 70 deg. F. and the relative humidity was 70 per cent.

Series 3

The purpose of these tests was to determine the effect upon flow of variations in the conditions of storage with regard to moisture. The conditions of storage which were included in the investigation were (1) air at 50 per cent relative humidity, (2) air at 70 per cent relative humidity, (3) fog, and (4) under water.

The specimens were 4 by 14-in. cylinders, 20 in number, 12 of which were loaded and 8 of which were used as unloaded controls.

The concrete was the same for all specimens, being composed of 1 part of cement to 5.67 parts by weight of Niles gravel. The mix data are given in the following table. The fineness modulus of the mixed aggregate was 5.61 and the water-cement ratio was 0.89.

TABLE 4—PROPORTIONS—SERIES 3

	Cement	Fine Aggregate	Medium Aggregate	Coarse Aggregate
Proportions by Weight	1.00	2.00	1.67	2.00
Fineness Modulus		3.48	5.99	7.40

The specimens were fog-cured for 28 days when those carrying a sustained stress were all loaded to 800 lb. per sq. in. For each of the four conditions of storage there were three loaded and two control specimens.

Series 4

The purpose of these tests was to determine the effect of the character of mineral aggregate upon flow. The aggregates which were included in the investigations included (1) Niles gravel, containing quartz, sandstones, granites, etc., (2) metamorphic sandstone from Oakland, Calif., (3) quartz, (4) basalt from Napa, Calif., (5) limestone from Santa Cruz, Calif., and (6) granite.

The specimens were 4 by 14-in. cylinders, 30 in number, of which 18 were loaded and 12 were used as controls.

The concrete was of the same richness of mix for all specimens, namely, 1 part of cement to 5.67 parts by weight of mixed aggregate. The water-cement ratio was uniformly 0.89 by volume.

For the gravel concrete, the mix and gradation were as indicated under Series 3. Other aggregates were crushed, screened in a variety of sizes, and then these sizes recombined to give a sieve analysis curve practically the same as that of the gravel. That is, the gradation of one mixed aggregate was the same as for all others. The fineness modulus of each mixed crushed aggregate was 5.75.

The consistency of the fresh concrete for the several aggregates, as determined by slump and flow tests was as shown in Table 5.

The specimens were fog-cured until loaded at the age of 28 days and were thereafter stored in air at 50 per cent relative humidity. For each aggregate, three specimens were loaded to a stress of 80 lb. per sq. in. and two specimens were employed as unstressed controls.

Series 5

The purpose of these tests was to determine the effect of steel reinforcement upon flow. The specimens are 10 by 20-in. cylinders, 4 in number, 2 of which are

TABLE 5—CONSISTENCY—SERIES 4

Aggregate	Slump., In.	Flow
Gravel	4	141
Sandstone	2¼	138
Quartz	4¼	156
Basalt	3	137
Limestone	2	137
Granite	3½	142

plain and 2 of which are reinforced with longitudinal bars and spiral hooping. The reinforcement consists of six ½-in. square deformed bars, and an 8-in. diameter spiral of No. 6 A. S. W. G. of 1¼ in. pitch. The steel is of structural grade and the ends of the longitudinal bars are ground flush with the ends of the cylinders. Considering the core area, the steel ratio for longtiudinal bars is 0.0298 and for spiral is 0.0133. Considering the entire cross-section of the cylinder, the steel ratio for the longitudinal bars is 0.0190.

The concrete is composed of 1 part of cement, to 2.64 parts Niles sand, to 2.32 parts crushed sandstone, all by volume. The fineness modulus of the combined aggregate was 5.22 and the water-cement ratio was 0.86. The slump was 2 in.

The specimens were cured in fog for 50 days and thereafter were stored in air at 50 per cent relative humidity. At the age of 60 days, one plain and one reinforced specimen were loaded. The stress in the plain concrete was 800 lb. per sq. in. and the stress in the reinforced concrete at the time of loading was calculated to have been 775 lb. per sq. in. on the full concrete area.

Series 6

The purpose of these tests was to determine the behavior of concrete under alternations of a maximum and a minimum sustained stress. Observations were made to determine the instantaneous and time yields each time the load was increased from minimum to maximum and the instantaneous and time recoveries each time the load was decreased from the maximum to the minimum.

The specimens are 4 by 14-in. cylinders, 30 in number, 18 of which are subjected to load alternations and 12 of which are unstressed control specimens.

The concrete was the same for all specimens. The mix by weight was composed of 1 part of cement to 5.68 parts of mixed aggregate. The mix data are given in Table 6. The fineness modulus of the combined aggregate was 4.86 and the water-cement ratio was 0.85. The slump was 4 in.

TABLE 6—PROPORTIONS—SERIES 6

	Cement	Monterey Sand	Niles Sand	Niles Gravel
Proportions by Weight	1	1.14	1.14	3.40
Fineness Modulus		1.74	2.90	6.72

The specimens were fog-cured for 28 days, when load alternations were begun. Thereafter half the specimens were stored in air at 50 per cent relative humidity and the remainder were stored under water.

For all specimens the maximum sustained stress was 800 lb. per sq. in. and the minimum sustained stress was 40 lb. per sq. in. A stress alternation or cycle consisted of a given period at the maximum stress followed by a like period at the minimum stress.

In Table 7 are given the lengths of stress cycle for the several groups of specimens.

TABLE 7—LENGTH OF STRESS CYCLES—SERIES 6

Storage Conditions	Length, Stress Cycle, Days	Group Designation	No. of Specimens	
			Loaded	Control
Air 50% humidity	2	C	3	3
Air 50% humidity	10 to 14	B	3	3
Air 50% humidity	60	A	3	
Water	2	C	3	3
Water	10 to 14	B	3	3
Water	60	A	3	

Series 7

The purpose of these tests was to study the effect of richness of mix and storage conditions upon flow and the effect of flow upon the compressive strength and the modulus of elasticity, that is, it was desired to learn how these properties of a concrete which had been subjected to a sustained stress for a considerable period of time would compare with those of an identical concrete not previously stressed.

The specimens were 6 by 12-in. cylinders, 64 in number, half of which were placed under continuous load at 28 days and the remainder of which were not stressed prior to the strength test.

Two concrete mixes were employed, a rich mix composed of 1 part cement to 3.6 parts by volume of combined aggregate, and a lean mix composed of 1 part cement to 6.3 parts of combined aggregate. For both mixes a constant slump of 3 to 4 in. was used, giving a water-cement ratio which varied slightly for each mix. For the rich mix the fineness modulus of the combined aggregate was 5.70 and the water-cement ratio was close to 0.63 by volume. For the lean mix the fineness modulus of the combined aggregate was 5.55 and the water-cement ratio varied from 0.95 to 1.02 by volume. Table 8 gives the proportions of the separate aggregates.

For the first 7 days all specimens were fog-cured. Thereafter half the specimens were stored under water and the remainder were stored in air at 50 per cent relative humidity. At 28 days, half of those stored in both air and water were loaded to 800 lb. per sq. in. The remainder, as noted above, were left unstressed.

TABLE 8—PROPORTIONS—SERIES 7

Cement-Aggregate, Ratio by Volume	Parts by Weight				
	Cement	Monterey Sand	Top Gravel	Medium Gravel	Coarse Gravel
1:3.6	1	0.66	0.66	1.34	1.34
1:6.3	1	1.26	1.26	2.24	2.24
Fineness Modulus		2.15	2.92	6.88	7.63

At the age of 5 months (a period of 4 months of sustained stress for those specimens which were loaded) half the number of specimens in each condition of storage and of each mix were tested in compression. At the age of 12 months, when the period of sustained stress was 11 months, the remaining specimens of each group were tested. At the ends of these periods the load was removed from the specimens which were subjected to sustained stress, and observations of instantaneous recovery were made. In the 5-months tests, as quickly as possible after unloading, the specimens were tested to failure, in compression, measurement of deformation being made with a compressometer as the loading progressed. In the 12-months tests, after unloading, the plastic recovery was made for four or five days before the elastic compression tests were made.

Testing Apparatus and Methods

Except for the elastic compression tests of Series 7, where an attached dial compressometer was employed, all changes in length have been measured with portable strain gages, those of Series 1 with a 20-in. gage of the multiplying-lever type, and those of all later series with 10-in. gage of the fulcrum-plate type. The latter type has been found particularly satisfactory.

For each specimen observations of changes in length have been made upon three longitudinal gage lines at the third points of the circumference. The reference marks are No. 50 drill holes into which are thrust the points of the strain gage. For Series 1, ½-in. diameter carbon steel plugs were grouted in the proper position into holes drilled in the specimens, the gage distance was punch marked with a templet, and the gage holes were drilled with a hand drill. For all later series ½-in. square brass plugs each threaded to receive a screw, were fastened to the interior of the molds in true position by means of screws passing through the molds. When the concrete had hardened sufficiently the screws were withdrawn and the molds were removed from the cylinders. Later in each brass plug was screwed a rustless steel insert with center hole forming the gage mark. This type of gage marker, consisting of the brass plug with hard-tempered, rustless steel insert, has proven so eminently satisfactory, not only because of its low cost for making and placing, but because of the precision with which the gage holes can be placed and aligned, and because of resistance to wear and corrosion, that attention is invited to its features.

For the specimens of Series 5, in addition to the three longitudinal gage lines with points in the circumferential surface, there are employed two additional

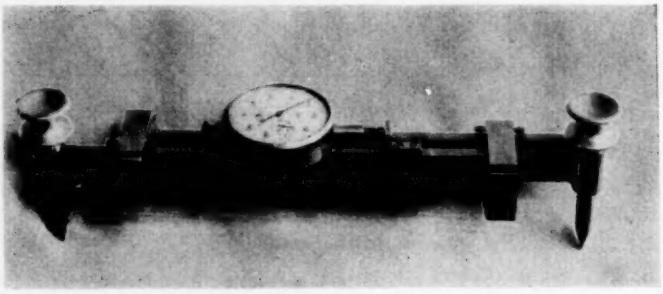

FIG. 1—FULCRUM PLATE STRAIN GAGE

sets of corresponding gage lines, one set of three with plugs set in the concrete at a depth of one inch from the surface, and the other set of three with gage holes in the longitudinal reinforcing bars at 1 in. depth.

In each loaded specimen the load has been maintained constant by means of one or more car springs held in compression by a system of rods and plates between which the specimen is clamped. The desired load is applied by mounting this apparatus in a testing machine with a test specimen in position, the load being indicated by the machine. In order to eliminate the disturbing influence of variable temperature, a small portable machine has been built specially for applying loads to these specimens, and this machine is placed in one of the constant temperature rooms where the specimens are stored. When a specimen is loaded, the nuts on the rods of the load-sustaining apparatus are brought to a bearing against the end plates. The load on the testing machine is then released and the compression in the car spring and in the specimen is maintained by tension in the rods. This method of maintaining stress has proved very satisfactory, since the specimens may be moved about and placed in any position after the load has once been applied, without disturbing stress conditions.

For each group of loaded specimens there is a corresponding group of unstressed or control specimens which is stored in the same conditions as the loaded specimens. Observations made upon these control specimens (made simultaneously with observations on loaded specimens) give changes in length due to causes other than stress, and this makes it possible to determine the flow which has occurred in the loaded specimens as a quantity separate from volume changes due to variations in moisture or temperature.

All specimens have been stored continuously at 70 deg. F. in specially insulated rooms heated by electricity and equipped with sensitive thermostats. Furthermore those stored in air have been placed in rooms where the humidity is controlled as well. Under normal conditions variations from the established standard do not exceed 1 deg. F. in temperature and 1 per cent in relative humidity.

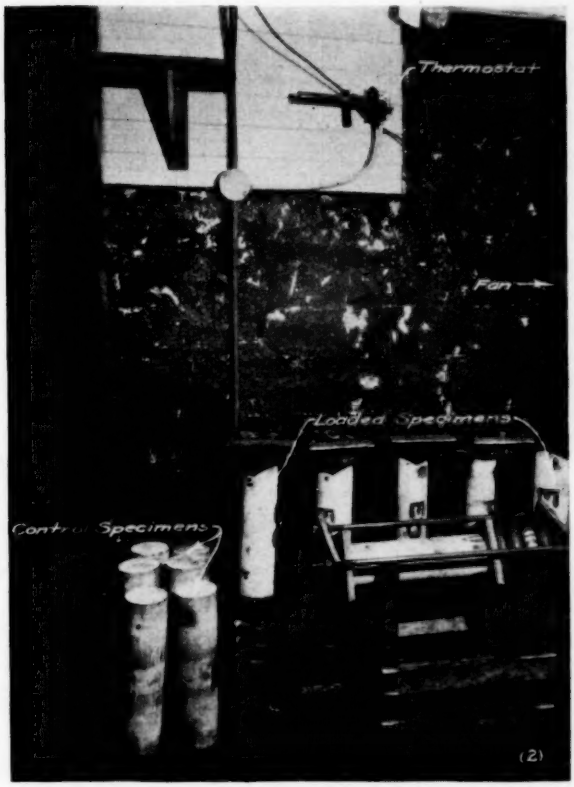

FIG. 2—SPECIMENS IN DRY AIR STORAGE, SERIES 2

Results of Tests

The data of the tests are presented in graphs exhibited as Figs. 4 to 26 inclusive, many of which show the time-flow relation. Each plotted point represents an average value determined by measurements made upon all specimens of a group. Though shrinkage measurements formed a large part of the test data, being taken on all control specimens, they are here eliminated from consideration, except for Series 5 (Fig. 15) in which there are reinforced columns, for which both time-shrinkage and time-flow curves have been drawn.

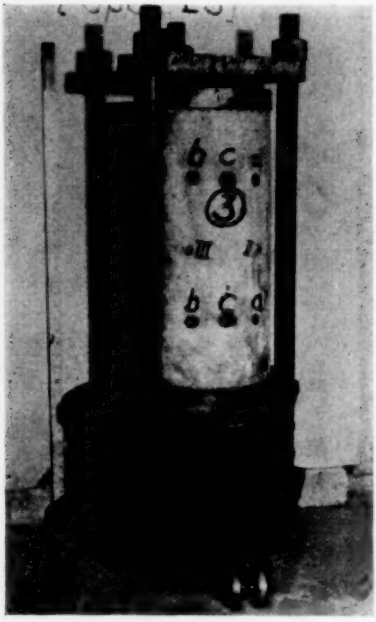

FIG. 3—REINFORCED SPECIMENS UNDER SUSTAINED LOAD, SERIES 5

For the time-flow curves the abscissa represent the time interval in days since the application of load and the ordinates represent flow under the action of sustained load as determined by deducting from the total change in length that due to shrinkage plus that which took place immediately upon application of the load. Examining the figures it will be noted that the flow curves have a characteristic shape, the rate of flow being most rapid immediately after the load is applied and gradually decreasing with time. This indicates that the law governing flow is asymptotic in character, the magnitude of flow gradually approaching a maximum value which depends upon many conditions.

An examination of the diagrams will show that for several conditions the state of flow equilibrium has been reached, but in general it appears that flow is still in progress, even after 3½ years of sustained load.

For several of the figures, in addition to flow curves, there are shown at the left end values of the instantaneous deformations or instantaneous recoveries to the same scale of ordinates as was employed in plotting the flow curves. These deformations and recoveries have been determined by strain-gage observations made immediately before and immediately after the application or release of load.

In Figs. 16 to 19, representing the effect of alternately applying and releasing load, the ordinates to the diagrams give the total deformation due to stress, beginning with the instant before the initial application of the load increment. That is, corrections for shrinkage have been made. The lengths of the full vertical lines represent instantaneous deformations or instantaneous recoveries for the given load increment (760 lb. per sq. in.), i. e., between minimum and maximum or maximum and minimum sustained stresses. The dash lines show the flows under the maximum sustained stress (800 lb. per sq. in.) or the time recoveries under the minimum sustained stress (40 lb. per sq. in.).

Effect Upon Flow of Richness of Mix, Gradation of Aggregate and Water-Cement Ratio—The results of the tests of Series 1 for which the specimens were subjected to a sustained compressive stress of 640 lb. per sq. in. at the age of 7 months, are shown by the diagrams of Fig. 4 and the results of the tests of Series 7, for which the specimens were placed under a sustained stress of 800 lb. per sq. in. at the age of 28 days are likewise represented by the diagrams of Fig. 5. The less precise instruments and methods employed for the earlier series explains the apparent irregularities of the curves of Fig. 4 as compared with those of Fig. 5. Consulting these two figures, it is seen that the general manner in which flow increases with time, as demonstrated by the time-flow curves is characteristically the same for the several groups of specimens represented, the rate of flow being most rapid immediately upon application of the load, and gradually decreasing with time. At the left end of each of the two figures are indicated the magnitudes of the instantaneous deformations which occurred when the load was applied and the instantaneous recovery when the load was released after the period of sustained load which was

approximately 5 months for the specimens of Series 1, and 11 months for the specimens of Series 7.

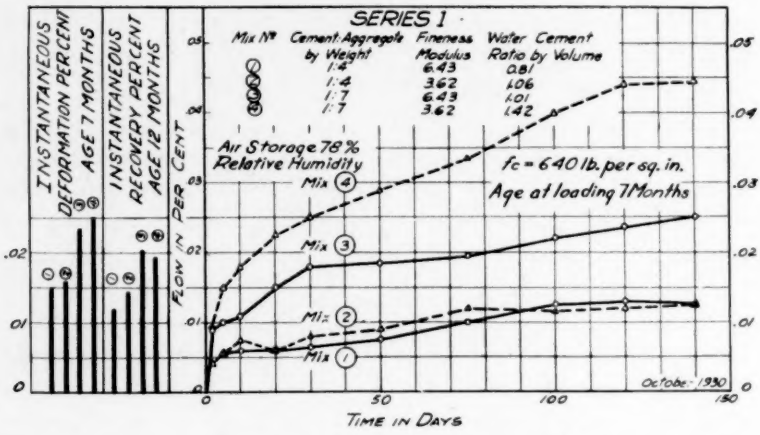

FIG. 4—EFFECT OF RICHNESS OF MIX AND GRADATION OF AGGREGATE UPON FLOW

From a study of the diagrams the following observations are made:

1. Regardless of age, richness of mix, gradation of aggregate, or conditions of storage, concrete subjected to sustained compressive stress plastically deforms for a long time.

2. The rate at which flow takes place under given constant conditions decreases with time, pointing to an eventual state of flow equilibrium.

3. Other things being equal, the leaner the mix, the greater the flow.

4. It appears that the greater the water-cement ratio, other factors remaining constant, the greater the flow, but that the water-cement ratio considered by itself is a lesser factor than is that of the gradation of the aggregate.

5. Other things remaining equal, the better graded the aggregate, or the higher the fineness modulus and the lower the void space, the less the flow.

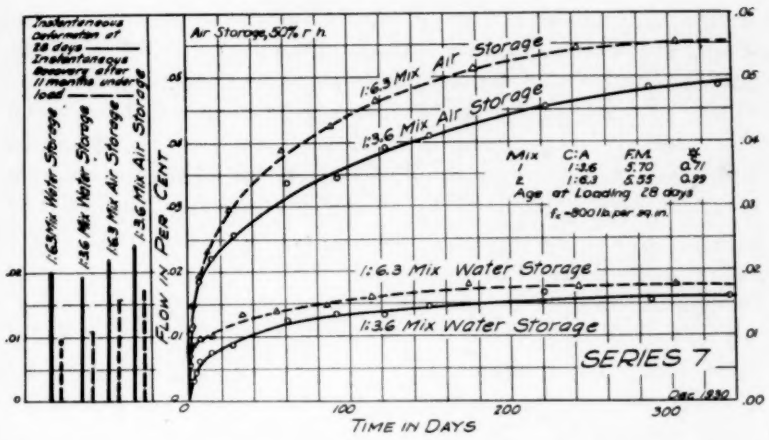
Instantaneous deformation at 28 days
Instantaneous recovery after 11 months under load
1:6.3 Mix Water Storage
1:3.6 Mix Water Storage
1:6.3 Mix Air Storage
1:3.6 Mix Air Storage
Air Storage, 50% r.h.
FLOW IN PER CENT
1:6.3 Mix Air Storage
1:3.6 Mix Air Storage
Mix C:A F.M.
1 1:3.6 5.70 0.71
2 1:6.3 5.35 0.99
Age at Loading 28 days
f'c = 3001b. per sq. in.
1:6.3 Mix Water Storage
1:3.6 Mix Water Storage
SERIES 7
Dec. 1930
TIME IN DAYS

FIG. 5—EFFECT OF RICHNESS OF MIX AND STORAGE CONDITIONS UPON FLOW

6. The instantaneous deformation is less for rich mixes than for lean ones.

7. It is not clear that either the gradation of the aggregate or the water-cement ratio influences the instantaneous deformation to a marked degree.

8. The instantaneous recovery is less than the instantaneous deformation, but this difference in general is not large nor is it clear that the magnitude of the instantaneous recovery is appreciably affected by the duration of the sustained load.

9. Under conditions of dry storage, the flow becomes substantially greater than the instantaneous deformation regardless of richness of mix, gradation of aggregate, or water-cement ratio.

10. It appears that under conditions of dry storage the flow is substantially greater than for the same quality of concrete kept perpetually wet. On the other hand, the instantaneous recovery is proportional to the instantaneous deformation, not being influenced by conditions of storage during the period of sustained load.

11. The total flow at the end of the 5-months sustained load for the lean mix of low fineness modulus of Series 1 (Fig. 4), is more than $\frac{1}{2}$ in. per 100 ft., and for the rich mixes approximately $\frac{1}{8}$ in. per 100 ft.

Effect of the Mineral Character of the Aggregate Upon Flow—The results of the tests of Series 4, made to determine the effect of the mineral character of the aggregate are shown by the curves of Fig. 6, one each for limestone, quartz, granite, gravel, basalt, and limestone specimens, and each plotted point representing the average of the given group. The concrete for all aggregates were identical as to richness of mix, water-cement ratio, gradation of aggregate, and conditions of curing. A sustained compressive stress of 800 lb. per sq. in. was applied at the age of 28 days and the specimens were stored in air of 50 per cent relative humidity. The record of flow observations has extended over a period of 2 years.

An examination of the figure shows the following:

12. Regardless of the mineral character of the aggregate, the rate of flow decreases with time, though at the end of two years of sustained load there is no evidence that flow has ceased.

13. The magnitude of the flow is appreciably affected by the character of aggregate, after 2 years being nearly three times as great for sandstone concrete as for limestone concrete, and increasing in the order, limestone, quartz, granite, gravel, basalt, sandstone.

14. As between aggregates of different mineral character, the magnitude of the instantaneous deformation occurring upon application of the sustained load is no criterion to the flow which will eventually take place. The instantaneous deformations for the several aggregates differ but slightly from one another, but the magnitudes of flow differ greatly.

15. The flow after two years of sustained load is 2 to 4 times (depending upon the aggregate) the instantaneous deformation which took place when the load was applied.

16. The rate of flow immediately subsequent to the application of load is generally, though not always, an index to the magnitude of flow after a long period of sustained load. (Among the aggregates investigated, gravel was the exception).

17. The magnitude of the flow produced by the sustained stress of 800 lb. per sq. in. over a period of two years varied from ½ in. per 100 ft. for limestone to 1½ in. per 100 ft. for metamorphic sandstone.

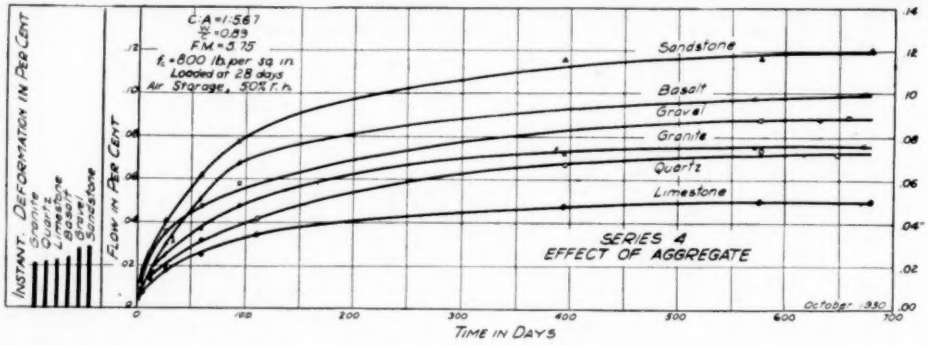

FIG. 6—EFFECT OF MINERAL CHARACTER OF AGGREGATE

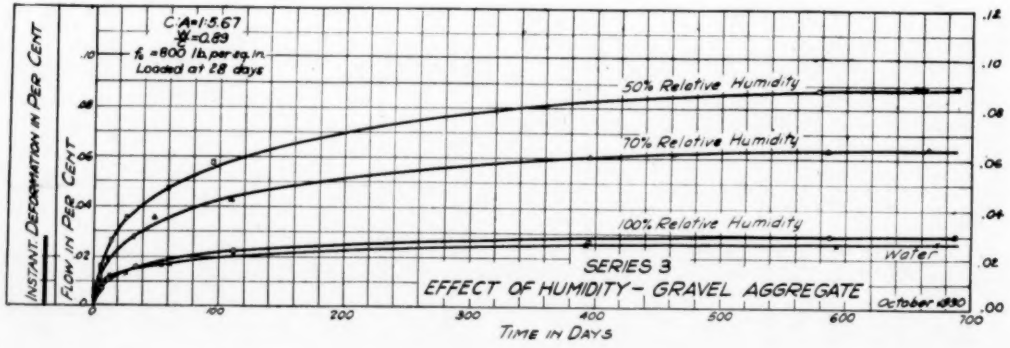

FIG. 8—EFFECT OF MOISTURE CONDITIONS, SERIES 3

Effect of Condition of Storage (As Regards Moisture) Upon Flow—The results of recent tests have demonstrated that the modulus of elasticity of concrete when wet is higher than for the same concrete when dry. In other words, under a given compressive load the instantaneous deformation in a specimen of given concrete saturated with water is less than in a specimen of the same concrete in an air-dry condition, this in spite of the fact that the two specimens may have been moist cured for a long period of time under identical conditions prior to loading.

It has been determined that an analogous condition exists in regard to flow under sustained stress, as is demonstrated by the results of Series 2 and 3, which are shown by Figs. 7 and 8. Further, not only does the analogy hold between fully saturated and air-dry concrete, but also as regards the degree of air-dryness as determined by the humidity of the surrounding air.

Considering Fig. 7, it is seen that there are shown time-flow curves for sustained stresses of 300, 600, 900 and 1200 lb. per sq. in., with ages at loading of 28 days and 3 months, the period of sustained load being about 3½ years. The full lines indicate the flow of groups of specimens stored in air of 70 per cent relative humidity and the dash lines indicate the flow of groups of identical specimens stored under water.

Considering Fig. 8, it is seen that there are shown time-flow curves for nearly two years, a curve for each of four different storage conditions, storage in air at 50 per cent, 70 per cent and 100 per cent (fog) relative humidity and storage under water. For these tests the specimens subjected to a sustained compressive stress of 800 lb. per sq. in. was applied at 28 days. By way of emphasis it perhaps should again be stated that the *flow* is the deformation due to stress alone and does not include shrinkage, which has been eliminated by means of measurements made upon control specimens.

From a study of the diagrams of the two figures the following observations are made:

18. Other things being equal, the flow of concrete stored in air is materially greater than the flow of concrete stored under water.

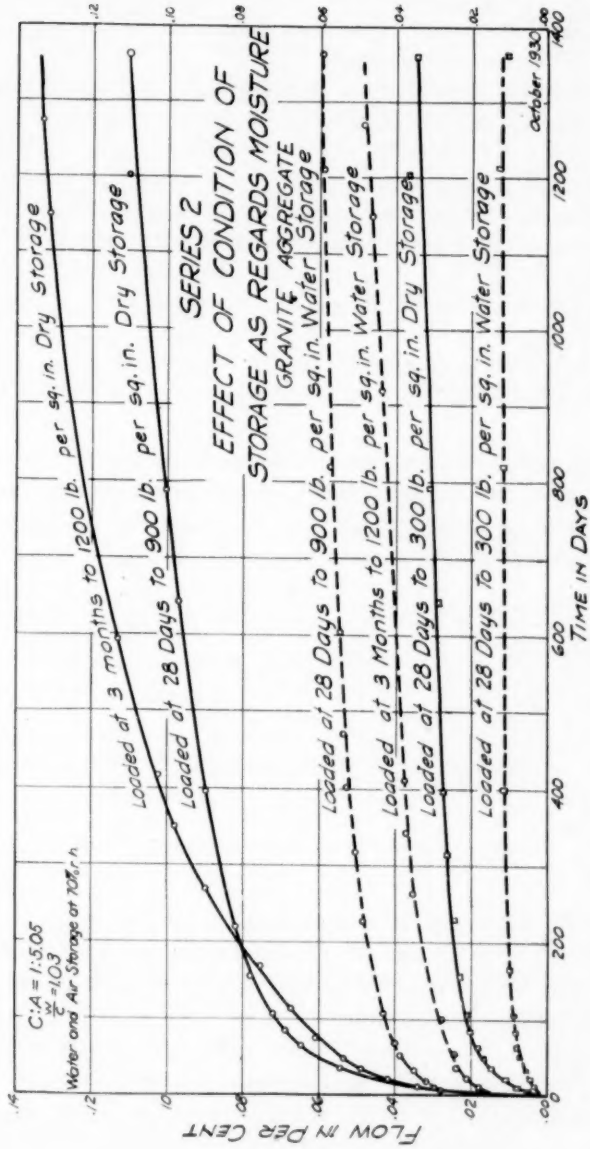

FIG. 7—EFFECT OF MOISTURE CONDITIONS, SERIES 2

and the lower the humidity of the air, the greater the magnitude of the flow. This appears to be true regardless of the intensity or duration of sustained stress and regardless of the age of the concrete at the time of loading.

19. The flows after about 2 years of sustained stress (Fig. 8) at 800 lb. per sq. in. for the several conditions of storage were as follows:

Storage in air at 50 per cent relative humidity	0.089 per cent.
Storage in air at 70 per cent relative humidity	0.063 per cent.
Storage in fog at 100 per cent relative humidity	0.030 per cent.
Storage under water	0.027 per cent.

20. The flow under 800 lb. per sq. in. stress for two years air storage at 50 per cent (Fig. 8) is three times the corresponding flow for concrete stored under water, and three times the instantaneous deformation which took place when the load was applied.

21. The age at loading as well as the intensity of stress and storage conditions as regards moisture has an important influence upon the flow as illustrated by the values of the flow after 3½ years shown on the following table.

TABLE 9—FLOW IN PER CENT AFTER 2½ YEARS, SUSTAINED LOAD—SERIES 2

Storage Condition	Age at Loading, 28 Days			Age at Loading, 3 Months		
	Stress 300 Lbs. per Sq. In.	Stress 600 Lbs. per Sq. In.	Stress 900 Lbs. per Sq. In.	Stress 600 Lbs. per Sq. In.	Stress 900 Lbs. per Sq. In.	Stress 1200 Lbs. per Sq. In.
Air 70% r. h.	0.034	0.069	0.111	0.062	0.081	0.134
Under water	0.012	0.035	0.059	0.018	0.036	0.048

Consulting the table, it is seen that for the range of ages at time loading and the range of sustained stresses represented, the flow in air at 70 per cent relative humidity is 2 to 3 times the corresponding flow under water.

The results of Series 6 and 7 also yield data on the effect of moisture conditions upon the elastic and plastic properties. In Fig. 19 are plotted the residual deformations, due to stress, that remain at the completion of each cycle of certain of the alternating load tests. The maximum stress of each cycle was 800 lb.

per sq. in. and the minimum was 40 lb. per sq. in. A study of the diagrams of this figure results in the following observations:

22. When a sustained compressive stress is alternately applied and released for given periods of time there is left a residual deformation at the end of each stress cycle (no load or low load condition). The magnitude of this residual deformation after a given number of stress cycles is not appreciably influenced by the length of such cycle (high-stress and low-stress periods being of equal duration) but is substantially greater for concrete under conditions of dry air (70 per cent r. h.) storage than under conditions of water storage.

23. After alternations extending over an 8-months period the residual deformation is three times as great for dry as for wet concrete.

In Figs. 20 and 21 are plotted stress-strain diagrams for concrete of two mixes at ages of 5 and 12 months, the ordinates being unit stresses and the abscissae being corresponding unit deformations which took place as the load was gradually applied. The full lines are stress-strain curves for groups of specimens not previously subjected to stress and the dash lines are stress-strain curves for groups which had been previously subjected to a sustained stress of 800 lb. per sq. in. for a period of 5 months for specimens tested at 6 months and for a period of 11 months for specimens tested at 12 months. From these figures it appears that:

24. The ratio of compressive stress to the instantaneous deformation produced thereby, all other conditions remaining the same, is less for a concrete which is dry than for one which is wet (as indicated by the slopes of the curves), regardless of the age of the concrete, the richness of mix, the magnitude of the stress, or whether or not the concrete has been previously subjected to sustained stress.

Effect of Age at Time of Loading Upon Flow—In Fig. 9 are plotted values of flow taking place in the several groups of Series 2 after 3½ years under sustained load. Consulting the figure it is observed that:

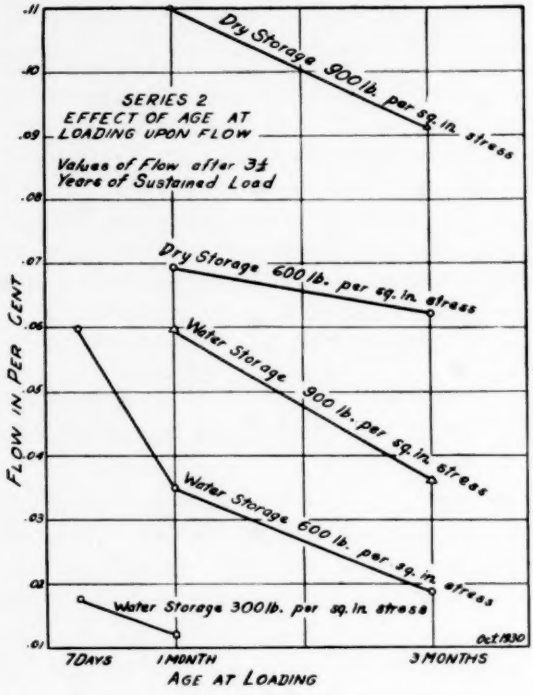

FIG. 9—EFFECT OF AGE OF LOADING UPON FLOW

25. The greater the age at the time of applying the sustained load, the less the flow. Thus the general effect of age is to increase resistance not only to those deformations which take place as the stress is applied but also to the plastic deformations which take place with time.

26. The rate of change in the magnitude of flow under given conditions is greater when the concrete is loaded at an early age than when loaded at a later age.

27. The effect of age at time of loading in reducing the rate of flow is more pronounced for high sustained stresses than for low ones.

Effect of Magnitude of Stress—The effect of the magnitude of the sustained compressive stress upon the flow resulting there-

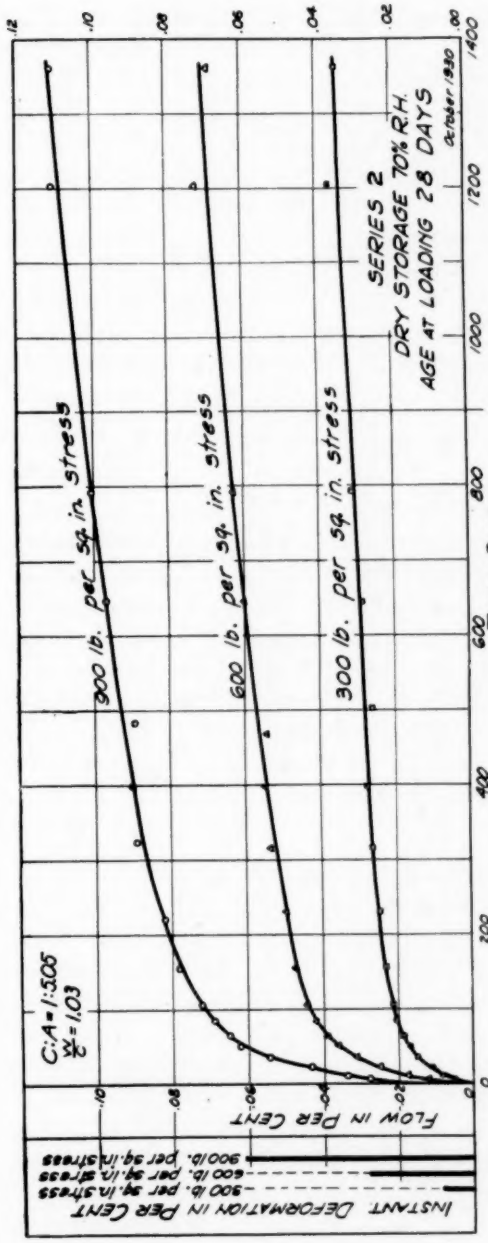
C:A = 1:5.05
W/C = 1.03
300 lb. per sq. in. stress
600 lb. per sq. in. stress
300 lb. per sq. in. stress
SERIES 2
DRY STORAGE 70% R.H.
AGE AT LOADING 28 DAYS
FLOW IN PER CENT
TIME IN DAYS
INSTANT DEFORMATION IN PER CENT
300 lb. per sq. in. stress
600 lb. per sq. in. stress
900 lb. per sq. in. stress

FIG. 10—EFFECT OF STRESS UPON AIR STORED SPECIMENS LOADED AT 28 DAYS—SERIES 2

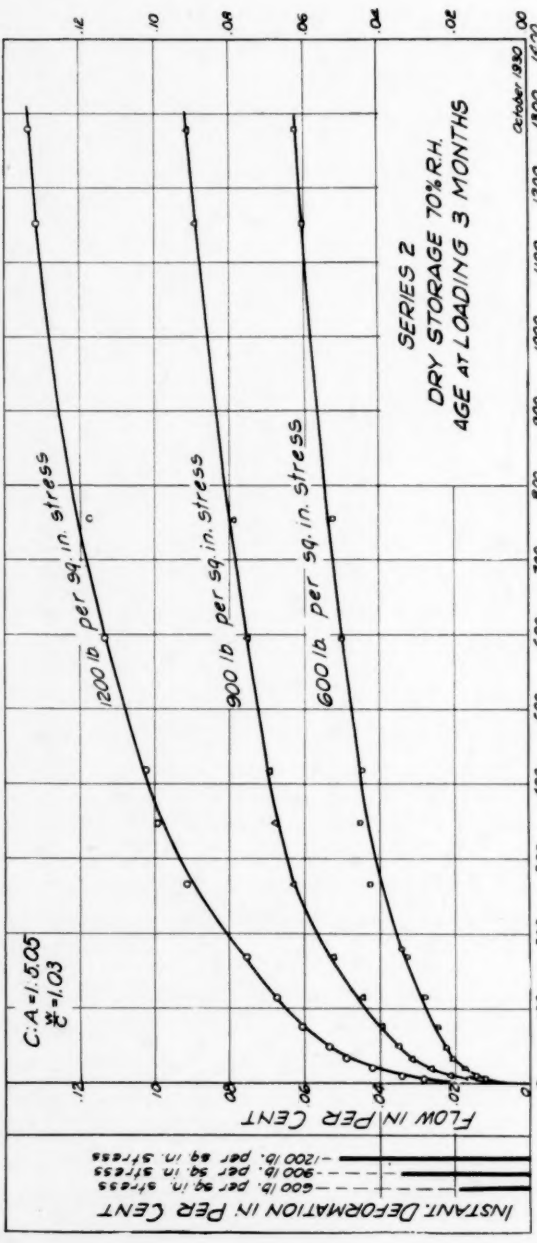
INSTANT. DEFORMATION IN PER CENT
FLOW IN PER CENT
TIME IN DAYS
SERIES 2
DRY STORAGE 70% R.H.
AGE AT LOADING 3 MONTHS
C:A=1.505
ε=1.03
1200 lb per sq. in. stress
900 lb per sq. in. stress
600 lb per sq. in. stress
600 lb. per sq. in. stress
900 lb. per sq. in. stress
1200 lb. per sq. in. stress

FIG. 11—EFFECT OF STRESS UPON AIR STORED SPECIMENS LOADED AT 3 MONTHS—SERIES 2

from has been determined by the tests of Series 2, where groups of specimens subjected to stresses varying 300 to 1200 lb. per sq. in. have been tested under wet and dry conditions, with loads applied at ages as early as 7 days and as late as 3 months. The results of these tests are shown graphically by the diagrams of Figs. 10 to 14, a separate figure for each condition of storage and age at time of loading. On each figure are shown time-flow curves for the sustained stresses of 300, 600, 900 and 1200 lb. per sq. in., these curves indicating the manner in which flow has taken place over a period of about 3½ years. At the left end of each figure heavy vertical lines show the magnitudes of corresponding instantaneous deformations which took place when the sustained stress was applied.

A study of these four figures results in a number of interesting observations:

28. Under conditions of dry storage, other things being equal, for any given period of sustained load in excess of about four months, the magnitude of the flow proportionally is somewhat greater for high sustained stresses than for low ones, but within the limits of the tests the departure from a linear stress-flow relation is no more marked than is the case for the instantaneous stress-strain ratio or secant modulus of elasticity. It seems reasonable to expect that the sustained stress-flow ratio, as the instantaneous stress-deformation ratio, will become more nearly constant within the range of working stresses as the age is increased.

29. Under conditions of wet storage, including ages of loading of 7 days, 28 days and 3 months, there is marked evidence that the sustained stress-flow relation, at any time after about four months of sustained load, is proportionally greater for high stresses than for low ones, the younger the age of concrete at time of loading, the greater the proportion. Thus by Fig. 12, for an age at loading of 7 days, the flow produced by a stress of 600 lb. per sq. in. is roughly 3½ times that produced by a stress of 300 lb. per sq. in., after any given period of sustained load between 100 and 1300 days. But for a loading age of 3 months (Fig. 14) the corresponding ratio for stresses of 1200 and 600 lb. per sq. in. is roughly 2½.

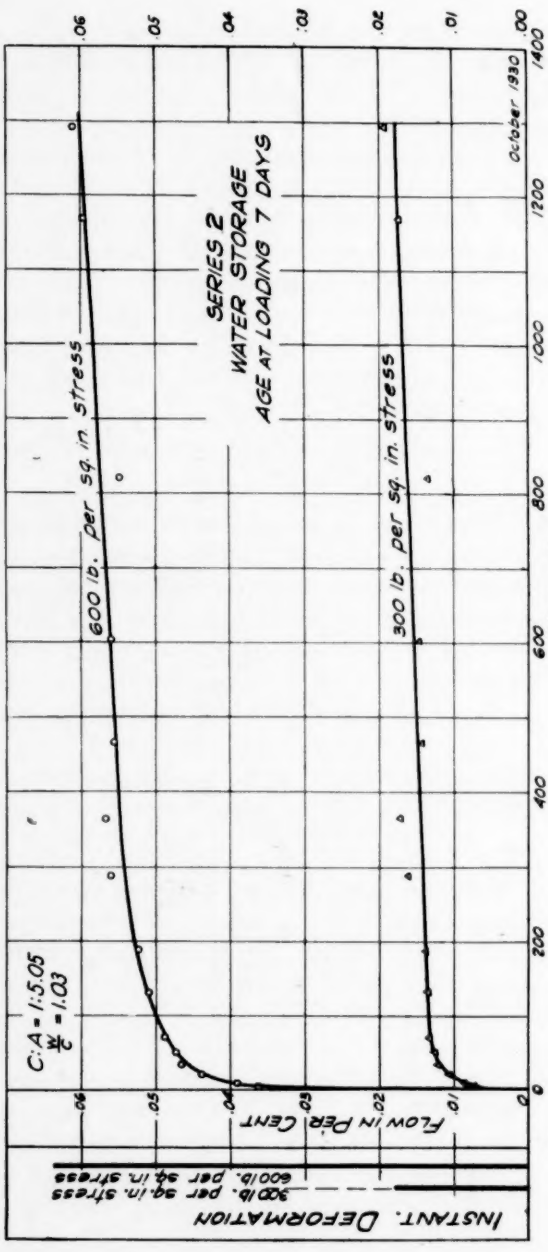

FIG. 12—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 7 DAYS—SERIES 2

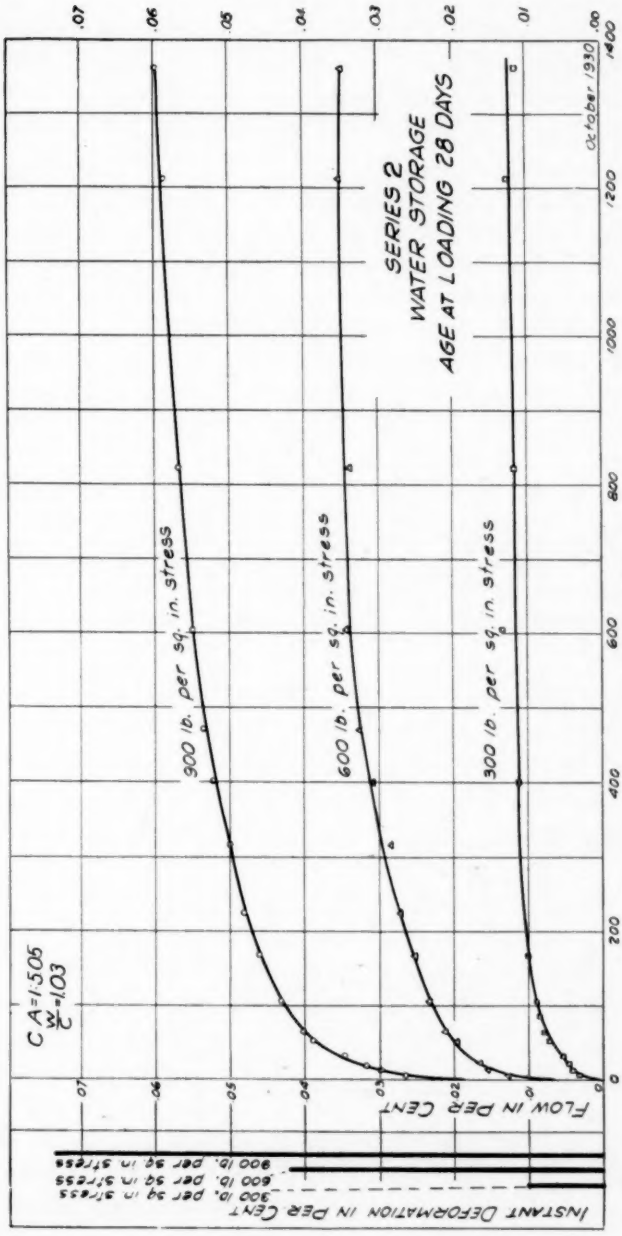
SERIES 2
WATER STORAGE
AGE AT LOADING 28 DAYS
300 lb. per sq. in. stress
600 lb. per sq. in. stress
300 lb. per sq. in. stress
C A=1.505
C=1.03
FLOW IN PER CENT
TIME IN DAYS
October 1930
INSTANT DEFORMATION IN PER CENT
300 lb. per sq in stress
600 lb. per sq in stress
800 lb. per sq in stress

FIG. 13—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 28 DAYS—SERIES 2

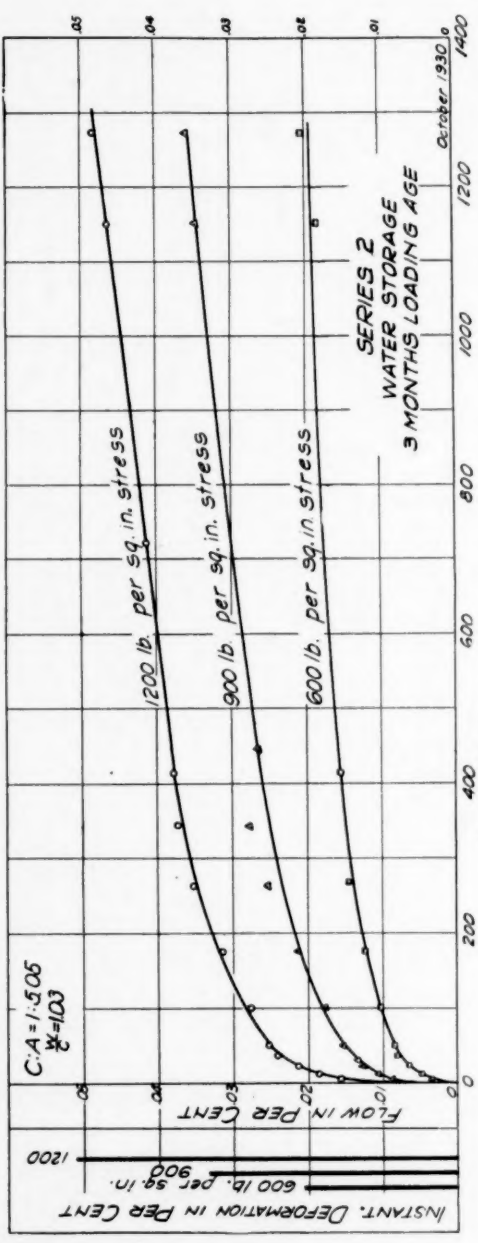

FIG. 14—EFFECT OF STRESS UPON WATER STORED SPECIMENS LOADED AT 3 MONTHS—SERIES 2

30. For all cases of water storage, regardless of age at time of loading and regardless of intensity of sustained stress, the flow roughly seems to be approaching as a limit the magnitude of the instantaneous deformation. There are some exceptions to this statement, but on the whole, the trend in this direction seems to be rather striking.

31. For all cases of air storage at 70 per cent relative humidity the flow under a sustained stress for 3½ years is substantially greater than the corresponding instantaneous deformation, the ratio of the former to the latter varying between about four for low stresses and about two for high stresses. Since for the higher stresses flow is increasing at a fairly rapid rate even after 3½ years, it would seem reasonable to expect that when flow equilibrium is finally established this ratio of flow to instantaneous deformation may reach a figure as high as four for all stresses. This is, of course, not a general prophecy but merely applies to the conditions of these particular tests.

The data of Series 2 and of Figs. 10 to 14 will be employed in a later discussion of the sustained modulus of resistance, total stress-deformation curves being shown for various periods of sustained loads.

Columns and the Effect of Reinforcement Upon Flow and Stress—The results of the tests of Series 5, made on short plain and reinforced column sections are given by the diagram of Fig. 15, instantaneous deformation being shown at the left and shrinkage and flow curves for plain and reinforced specimens being plotted one above the other.

The tests were begun when the specimens were 60 days old, the previous storage having been 50 days in fog and 10 days in air at 50 per cent relative humidity. The stress was 800 lb. per sq. in. in the plain concrete at the time of loading. The increment of stress placed in the concrete of the reinforced section at the time of loading was 775 lb. per sq. in. (assuming the modulus of elasticity of the steel as 30,000,000 lb. per sq. in.) but the actual stress in both the longitudinal steel and the concrete is not precisely known because observations of shrinkage were not begun upon control specimens until the sustained loads were applied.

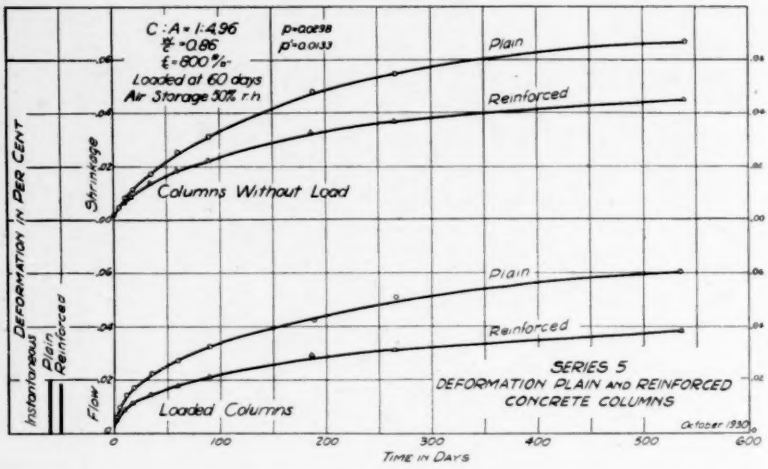

FIG. 15—DEFORMATION OF PLAIN AND REINFORCED CONCRETE COLUMNS—SERIES 5

Considering the curing conditions prior to loading it seems probable that a slight shrinkage had taken place during the 60-days curing period, in which case there was a small initial tension in the concrete and an initial compression in the steel. The longitudinal bars were ground flush with the plane ends of the columns immediately prior to applying the load, and the load was transmitted to the column through heavy steel plates bearing directly against reinforcing steel and concrete; hence stress was transmitted directly from load to reinforcement.

Considering the diagrams of Fig. 15, the following observations are made:

32. The effect of reinforcement is appreciably to reduce both shrinkage and flow.

33. There is a striking similarity between the shrinkage and flow diagrams, both for plain and reinforced specimens, though in general the shrinkage is somewhat greater than the flow.

34. The shrinkage after 18 months in air at 50 per cent relative humidity and the flow produced by a corresponding period of sustained compressive stress are each about ¾ in. per 100 ft. for plain columns and ½ in. per 100 ft. for reinforced columns.

35. For plain columns the combined flow and shrinkage over the loading period of 18 months is approximately six times greater than the instantaneous deformation which took place when the load was first applied.

36. Similarly, for reinforced specimens the combined flow and shrinkage is four times the instantaneous deformation.

37. By computations it can be shown that the compressive stress in the steel produced by load and shrinkage during the 18 months over which the observations extend is made up of the following:

Stress due to instantaneous deformation	5700 lb. per sq. in.
Stress due to flow	11400 lb. per sq. in.
Stress due to shrinkage	13200 lb. per sq. in.
Total change in steel stress	30300 lb. per sq. in.

Since the steel was structural grade, it appears that the stress was near the yield point. Obviously had the load been applied at an early age, as will often be the case in service structures, the yield point would certainly have been reached before 18 months.

38. Assuming the remainder of the load not carried by the longitudinal steel as being uniformly distributed over the full cross-section of the concrete, during the 18 months of sustained load, the sustained stress in the concrete was reduced from 775 lb. per sq. in. to 300 lb. per sq. in.

39. From observations of lateral deformation not here shown, there is computed as existing in the spiral hooping a compressive stress of 13800 lb. per sq. in. for both unloaded and loaded specimens, there being lateral shrinkage but no evidence of an appreciable lateral flow.

40. It seems evident that the yield point of the steel should be a factor in the design of columns carrying a large proportion of dead load when surrounding conditions are such as to produce shrinkage.

Effect of Alternating Periods of Sustained Stress—Figs. 16, 17 and 18 show graphically the results of the tests of Series 6, where specimens were subjected to alternating periods of high and low sustained compressive stress. For this series the specimens were loaded at the age of 28 days, first to a sustained stress of 800 lb. per sq. in. for a given period, then the stress was reduced to 40

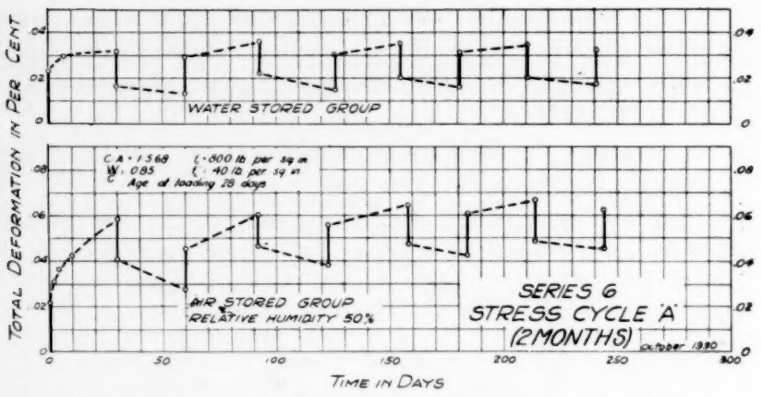

FIG. 16—EFFECT OF STRESS ALTERNATIONS—2 MONTHS CYCLE

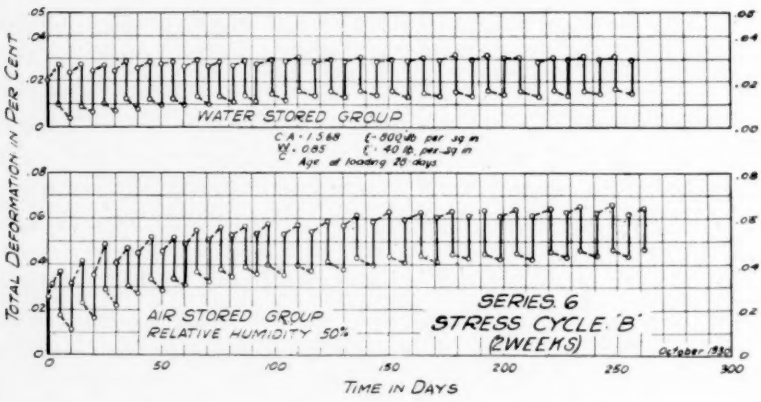

FIG. 17—EFFECT OF STRESS ALTERNATIONS—2 WEEKS CYCLE

lb. per sq. in. for the same length of time, and so the process of alternating high and low sustained stresses was repeated, the length of stress cycle varying between three groups of specimens but remaining substantially constant for the specimens of each group. The length of Stress Cycle A, for which the results are shown by the diagrams of Fig. 16, was two months, that is, each alternation consisted of one month at the stress of 800 lb. per sq. in., followed by one month at 40 lb. per sq. in.; the length of

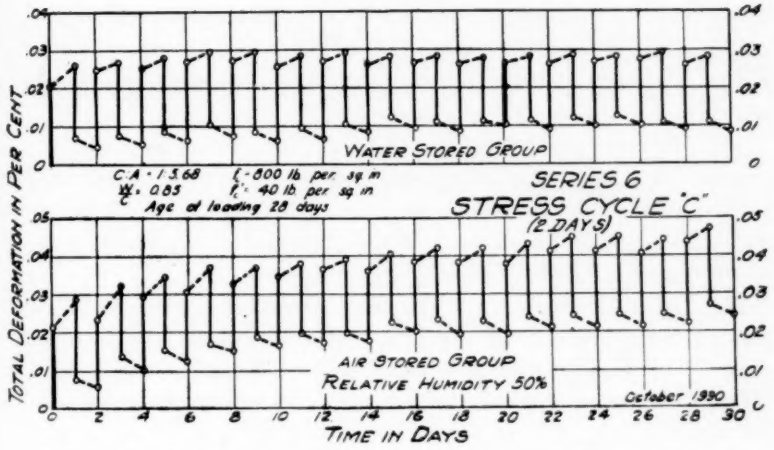

FIG. 18—EFFECT OF STRESS ALTERNATIONS—2 DAYS CYCLE

Stress Cycle B (Fig. 17) at first was 10 days, but later was two weeks; and the length of Stress Cycle C (Fig. 18) was 2 days.

Consulting the three figures, the following significant facts become evident:

41. The instantaneous deformation which took place at the time of first applying the load is greater than any succeeding value of the deformation, and greater than any value of the instantaneous recovery, demonstrating that a permanent set takes place upon the initial application of the load which is not later recovered. This is true regardless of the duration of the stress cycle.

42. Generally speaking, for specimens stored under water, the instantaneous deformations (after the first) and the instantaneous recoveries become somewhat less as the number of stress cycles is increased, but the decrease in these values is not large.

43. As a usual thing, for specimens stored in air there is little if any change in the magnitude of instantaneous deformations (after the first) with the number of alternations of stress, nor are there appreciable variations in the magnitude of the instantaneous recovery with number of alternations.

44. After the initial application of the load, there is general agreement between an instantaneous deformation and the

succeeding instantaneous recovery, regardless of the duration of stress cycle or condition of storage.

45. The rate of flow and of plastic recovery decreases as the number of stress cycles increases.

46. In general, the rate of flow for a given stress cycle is greater than the rate of plastic recovery for the same stress cycle. This characteristic becomes less pronounced as the number of cycles increases; it is more pronounced for specimens in air storage than for those in water storage.

47. Deformations and recoveries, whether instantaneous or plastic, for a given cycle are greater for specimens stored in air than for corresponding specimens stored in water.

Let the "stress deformation" be defined as the decrease from the original length (under minimum load) to that at the middle of any stress cycle at the instant preceding the release of maximum load. Also let "residual deformation" be defined as the decrease from the original length to that at the end of any stress cycle at the instant preceding the application of the maximum load.

48. Both stress deformations and residual deformations increase with the number of stress cycles. There are some indications that the magnitude of the residual deformation continues to increase after the stress deformation has become practically constant.

49. Both stress deformations and residual deformations increase at a more rapid rate with the number of cycles and become substantially larger for specimens in air storage than for those in water storage.

In Fig. 19 are plotted the residual deformations for water-stored and air-stored groups of specimens subjected to 2 months (Cycle A) and 2 weeks (Cycle B) alternations of stress. Consulting the figure it appears that:

50. Under similar stress conditions the residual deformation is approximately the same at the end of a given period of time regardless of the length of the stress cycle. This would also apply to the stress deformations. It is true for specimens in water storage as well as for those in air storage.

51. For Cycles A and B the residual deformations after 8 months of alternating load are approximately 3 times as great

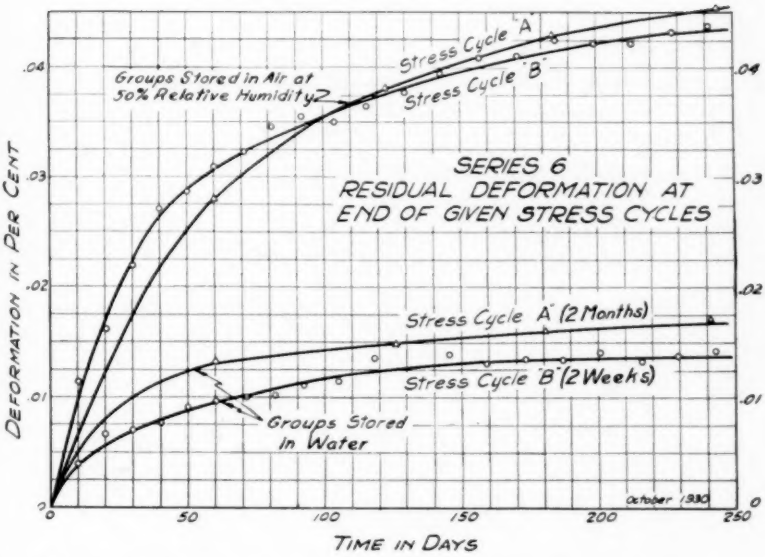

FIG. 19—EFFECT OF ALTERNATING STRESS UPON RESIDUAL DEFORMATIONS

with air-stored as with water-stored specimens; the corresponding stress deformations are twice as great for air-stored as for water-stored specimens.

Effect of Flow Upon Strength and Modulus of Elasticity—Table 10 and Figs. 20 and 21 give the results of tests upon specimens of Series 7 made to determine the effect of flow upon compressive strength and modulus of elasticity. For this series half of the specimens were placed under a sustained compressive stress of 800 lb. per sq. in. at the age of 28 days, while the other half without stress were maintained under corresponding conditions. At the ages of 5 and 12 months, specimens from each of these two groups were tested in compression under a gradually increasing load, the sustained load having been released prior to the application of the ultimate load.

Consulting Table 10 and the diagrams of Figs. 20 and 21, the following facts are in evidence:

52. The effect of flow under the action of a sustained load is to increase the compressive strength over that of identical con-

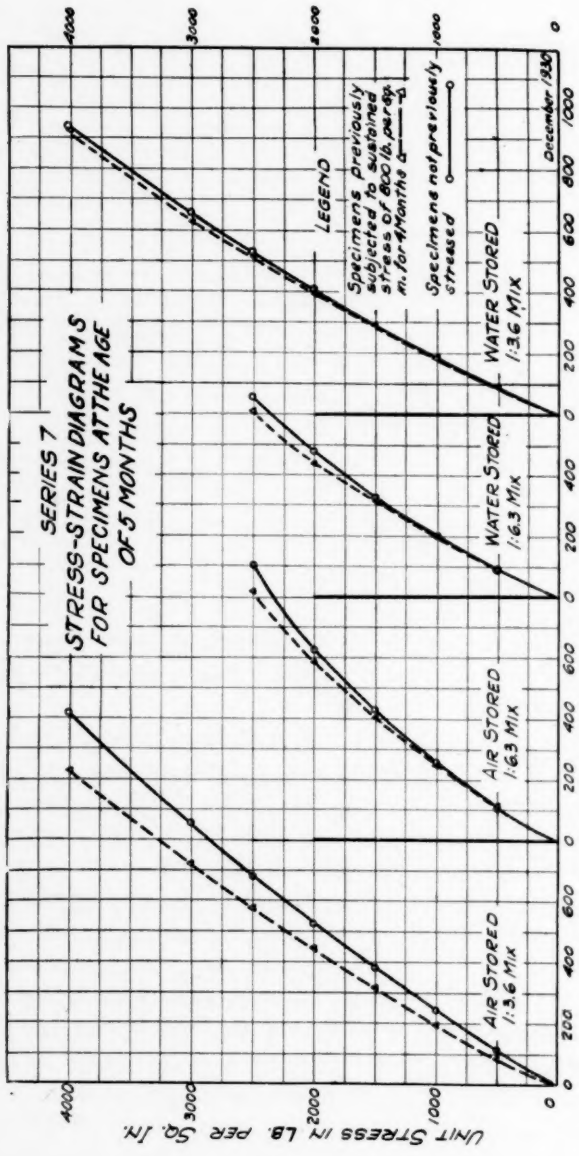

FIG. 20—SERIES 7

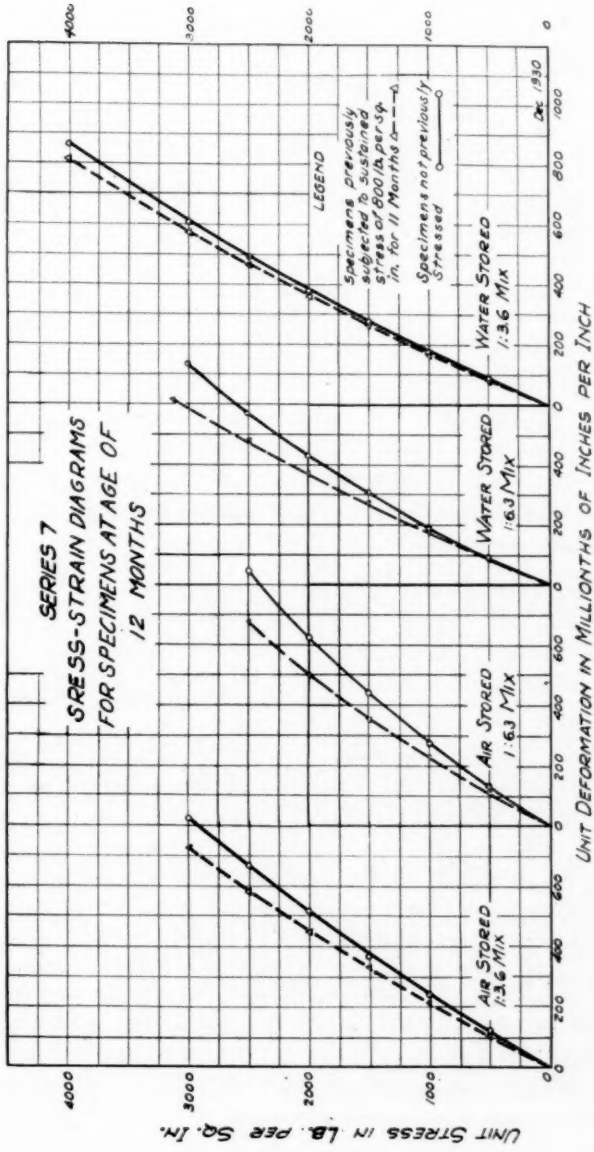

FIG. 21—SERIES 7

TABLE 10—EFFECT OF FLOW UPON STRENGTH—SERIES 7

Age, Months	Storage	Compressive Strength, Lb. per Sq. In.			
		Rich Mix (1:3.6)		Lean Mix (1:6.3)	
		Stressed	Unstressed	Stressed	Unstressed
5	air	5340	5110	3260	3250
5	water	6220	6020	3460	3560
12	air	5200	4770	3660	3460
12	water	5940	5790	4220	4150

crete not previously stressed, but the amount of the increase in compressive strength due to flow is small. There are some indications that the longer the period of sustained stress, the greater this increase becomes.

53. The influence of moist storage conditions after the age of 28 days upon the compressive strengths at later ages is manifest, since at both 5 and 12 months the water-stored specimens exhibit substantially greater strengths than do corresponding air-stored specimens.

54. The effect of flow under the action of sustained stress is appreciably to increase the instantaneous stress-deformation ratio or the secant modulus of elasticity over that exhibited by identical concrete not previously stressed. This is demonstrated by the curves of Figs. 20 and 21, the dash lines for specimens in which flow had occurred having a substantially greater slope than the full lines for specimens not previously stressed. This is true for high stresses as well as for low ones.

54. The effect of flow is to render the instantaneous stress-deformation relation or modulus of elasticity more nearly constant for all stresses, as indicated by the relative straightness of the corresponding stress-strain diagrams of the figures.

55. The longer the period of sustained load, the greater the increase in the instantaneous modulus of elasticity over that for

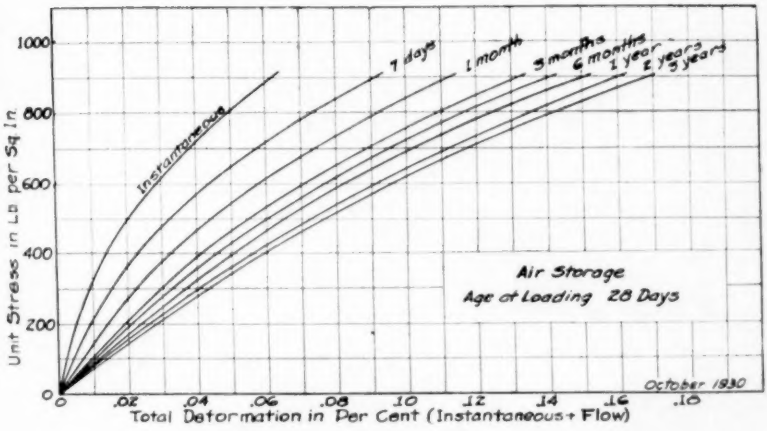

FIG. 22—STRESS-DEFORMATION DIAGRAMS—SERIES 2—AIR STORAGE, LOADED AT 28 DAYS

corresponding specimens not previously stressed. This time effect is more pronounced for the lean mix than for the rich one.

56. The greater the age, the more nearly constant for all stresses becomes the instantaneous modulus of elasticity. This is true for air storage as well as for water storage, and is true for specimens not previously stressed as well as for those previously subjected to sustained load.

57. The instantaneous modulus of elasticity is higher for specimens in wet storage than for corresponding specimens in dry storage. This is true regardless of age, mix, or previous stress condition.

Modulus of Resistance—Perhaps a clearer idea of the effect of sustained compressive stress may be gained by considering the relation between unit sustained stress and total unit deformation, including the instantaneous deformation and flow. With this thought in mind, the diagrams of Figs. 22, 23 and 24 have been prepared from the data of Series 2, the ordinates being unit sustained stresses and the abscissae being unit total deformations. Thus each diagram indicates the stress-strain relation for a group of specimens under the action of stress sustained for the period indicated on the diagram, and these time-stress-deformation curves have been collected into sets, each set for a given condition

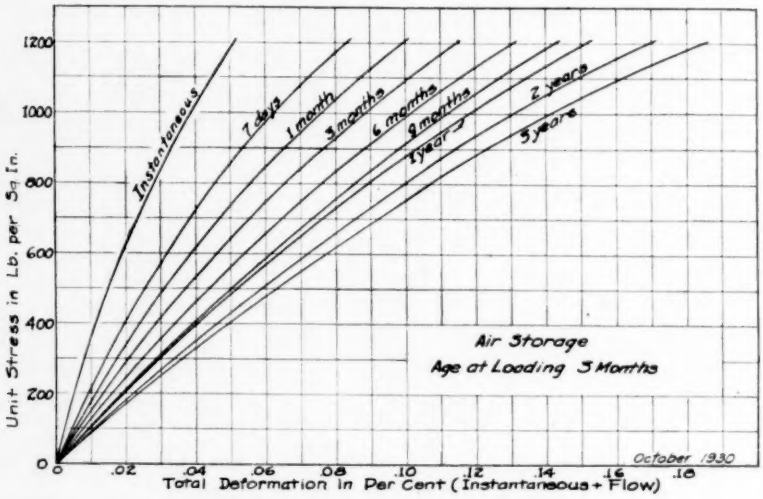

FIG. 23—STRESS-DEFORMATION DIAGRAMS—SERIES 2—AIR STORAGE, LOADED AT 3 MONTHS

of storage and age at application of sustained load. The points employed in plotting these diagrams have been taken from time-flow curves. Hence the curves of Figs. 22-24 pass through plotted points, and for this reason the points are not shown.

Modulus of resistance has been defined as the ratio of unit sustained stress to unit total deformation. Using the diagrams of Figs. 22-24 as a basis, values of the modulus of resistance for various intervals of sustained stress (800 lb. per sq. in.) up to 3 years. The results in millions of pounds per sq. in. are given in Table 11. The "instantaneous" modulus of resistance is merely the secant modulus of elasticity in the usual terminology.

Similarly computed values of the modulus of resistance for Series 3 and 4 are given in Table 12.

A study of the diagrams and tables indicates that the following relations obtain:

58. The modulus of resistance for concrete under constant sustained compressive stress decreases with time, at first rapidly and then more and more gradually for a long period. Under normal working stresses the sustained modulus of resistance

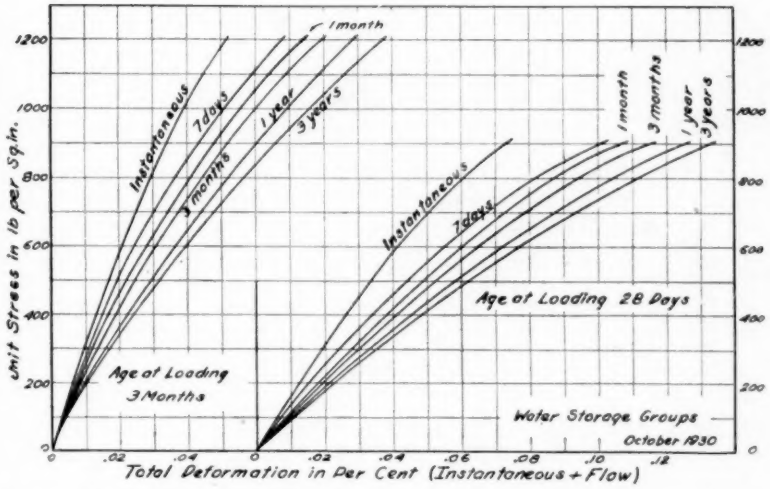

FIG. 24—STRESS-DEFORMATION DIAGRAMS—SERIES 2—WATER STORAGE, LOADED AT 28 DAYS AND 3 MONTHS

TABLE 11—VARIATION OF MODULUS OF RESISTANCE WITH PERIOD OF SUSTAINED LOAD—SERIES 2

Period of Sustained Stress	Modulus of Resistance in Millions of Lb. per Sq. In.*			
	Air Storage		Water Storage	
	Age at Loading, 28 Days	Age at Loading, 3 Months	Age at Loading, 28 Days	Age at Loading, 3 Months
Instantaneous	1.63	2.72	1.26	2.76
7 Days	1.09	1.64	0.93	2.20
30 Days	0.87	1.37	0.86	1.95
3 Months	0.73	1.17	0.81	1.81
6 Months	0.68	1.01	....	....
1 Year	0.64	0.74	0.74	1.53
2 Years	0.59	0.67	0.71	1.48
3 Years	0.56	0.62	0.70	1.42

*At a stress of 800 lb. per sq. in.

eventually becomes but a fractional part of the instantaneous value that was obtained when the load was applied. (This fractional part under certain conditions was less than one fifth.)

TABLE 12—VARIATION OF MODULUS OF RESISTANCE WITH PERIOD OF SUSTAINED STRESS—SERIES 3 AND 4

Period of Sustained Stress	Modulus of Resistance in Millions of Lb. per Sq. In.*								
	Gravel				Sandstone	Basalt	Granite	Quartz	Limestone
	50% r. h.	70% r. h.	100% r. h.	Water	50% r. h.	50% r. h.	50% r. h.	50% r. h.	50% r. h.
Instant.	2.91	2.91	2.91	2.91	2.81	3.56	3.90	3.72	3.56
1 Month	1.26	1.47	1.93	1.93	1.13	1.24	1.70	1.84	1.93
3 Months	0.97	1.17	1.65	1.72	0.76	0.90	1.20	1.37	1.48
6 Months	0.84	1.03	1.54	1.62	0.65	0.79	0.99	1.09	1.25
9 Months	0.75	0.96	1.48	1.55	0.61	0.73	0.90	0.98	1.20
12 Months	0.73	0.92	1.44	1.52	0.58	0.70	0.86	0.91	1.15
18 Months	0.70	0.88	1.42	1.47	0.55	0.67	0.83	0.87	1.11

*At a stress of 800 lb. per sq. in.

59. Under like conditions the modulus of resistance of concrete loaded at an early age always remains less than the corresponding modulus for the same quality of concrete loaded at a later age.

60. Other things remaining equal, the conditions of storage as regards moisture have an important bearing upon the magnitude of the modulus of resistance. Regardless of the period of sustained load, the sustained modulus of resistance is higher for a concrete in water storage than in air storage and is higher for a concrete surrounded by air of high humidity than for one surrounded by air of low humidity.

61. The mineral character of the aggregate influences to a marked degree not only the instantaneous modulus of elasticity, but even more so the sustained modulus of resistance. After 18 months of sustained load (Table 12) the modulus for limestone concrete is double that for sandstone concrete. The corresponding ratio for the instantaneous modulus is $\frac{4}{3}$.

Plastic Recovery—In considering the effect of alternating loads the question of plastic recovery received some attention. It was shown by diagrams that following each release of load there was an instantaneous recovery and that this was followed by a plastic recovery which increased with time.

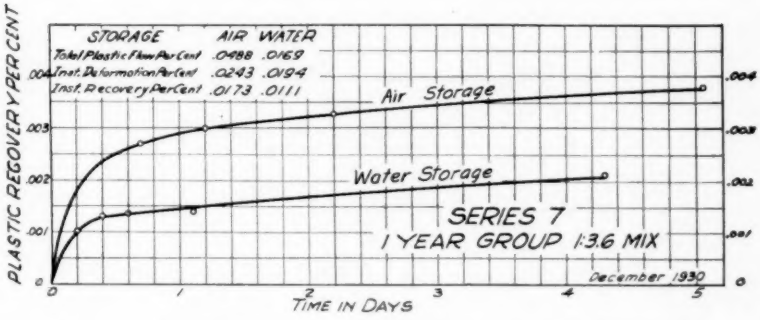

FIG. 25—PLASTIC RECOVERY—SERIES 7

The manner in which this plastic recovery takes place, and its magnitude when plastic equilibrium has been established, as compared with the preceding flow, are of interest.

Fig. 25 has been prepared by plotting observations of plastic recovery made on specimens of Series 7 which were tested in compression at the age of one year, after having been under load for 11 months. After the sustained load was released the specimens of this group were observed at intervals for several days in order to determine the general shape of the recovery curves for both air and water-storage.

Similarly, Fig. 26 shows residual deformation and plastic recovery curves for Group C of the alternating load tests of Series 6. The specimens of this group after fifteen 2-day alternations of stress were allowed to rest for a period of about 7 months, observations of changes of length being made periodically for both wet and dry groups. The full line curves of Fig. 26 are drawn through points which represent the residual deformations which were measured at the end of each successive 2-day cycle of stress, the measurement having been made just before another application of the load. The dash lines indicate the plastic recovery during the 7 months following the release of the last alternating load during which period the specimens were at rest.

An examination of Figs. 25 and 26 shows that:

62. The general shape of the time-recovery curves is much the same as for corresponding time-flow curves, but the rate of

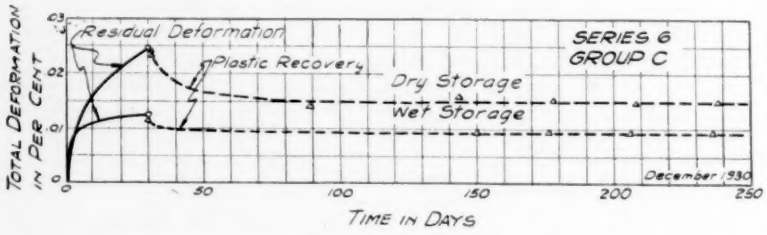

FIG. 26—PLASTIC RECOVERY—SERIES 6, GROUP C

recovery is comparatively small, and the state of plastic equilibrium is reached in a comparatively short time.

63. Plastic recovery for concrete in water storage is considerably less than the corresponding plastic recovery in air storage.

64. The plastic recovery as compared with the flow is a comparatively small quantity, much of it taking place within a few hours after the release of load. For example, Fig. 25 shows that the plastic recovery in 1 day is ¾ that in 5 days. Fig. 26 indicates that there is no further recovery after perhaps 2 months. Also, from Fig. 25 the plastic recoveries are respectively 0.004 and 0.002 for the dry and wet groups; while the corresponding flows for these groups, due to previously sustained load, were 0.048 and 0.016 per cent, values roughly ten times as great.

Summary of Flow Values—In order to render easy a comparison of the principal data concerning deformations produced by sustained stress which have been presented in the form of tables and graphs in the preceding pages, Table 13 has been prepared. In the Column headed "Instant Deformation," are given, in per cent, values of the deformations which took place immediately upon application of the sustained compressive stress stated under "Conditions." In the last column are recorded the plastic deformations or flows which were produced by the sustained stress subsequent to its application and during the time interval stated under "Conditions."

TABLE 13—SUMMARY OF OBSERVED DEFORMATIONS

Conditions	Instant. Deform. Per Cent	Flow Per Cent
Series 2—Period of Sustained Stress—1300 Days		
Dry storage, 28-day age at loading,		
Air at 70% relative humidity.		
Stress 900 lb. per sq. in.	0.06	0.11
600 do	0.03	0.07
300 do	0.01	0.03
Dry storage, 3 months age at loading		
Stress 1200 lb. per sq. in.	0.05	0.13
900 do	0.04	0.09
600 do	0.02	0.06
Water storage, 28 days age at loading		
Stress 900 lb. per sq. in.	0.07	0.06
600 do	0.04	0.04
300 do	0.01	0.01
Water storage, 3 months age at loading		
Stress 1200 lb. per sq. in.	0.05	0.05
900 do	0.03	0.04
600 do	0.02	0.02
Water storage, 7 day age at loading		
Stress 600 lb. per sq. in.	0.06	0.06
300 do	0.02	0.02
Series 3—Period of Sustained Stress—650 Days		
Sustained stress 800 lb. per sq. in.		
Air at 50% relative humidity	0.03	0.09
70% do	0.03	0.06
100% do	0.03	0.03
Water	0.03	0.03
Series 4—Period of Sustained Stress—650 Days		
Air at 50% relative humidity		
Stress 800 lb. per sq. in.		
Sandstone aggregate	0.03	0.12
Basalt aggregate	0.02	0.10
Granite aggregate	0.02	0.08
Quartz aggregate	0.02	0.07
Limestone aggregate	0.02	0.05
Series 5—Period of Sustained Stress—500 Days		
Stress in concrete at time of loading, 800 lb. per sq. in.		
Age at loading, 2 months.		
Storage 50% relative humidity		
Plain	0.02	0.06
Reinforced	0.02	0.04
Series 7—Period of Sustained Stress—300 Days		
Air at 50% relative humidity		
1:6.3 mix	0.02	0.06
1:3.6 mix	0.02	0.04
Water		
1:6.3 mix	0.02	0.02
1:3.6 mix	0.02	0.02

PART 2—SUMMARY OF VARIOUS PUBLISHED DATA CONCERNING FLOW OF CONCRETE

Following are brief abstracts of what are regarded as some of the more important published reports describing tests for which the phenomenon of gradually increasing deformations under the action of sustained loads has been observed and the results have been discussed. These include laboratory tests upon specimens, in the form of reinforced beams and slabs subjected to transverse loading and cylinders subjected to axial compressive stress, as well as field observations upon beams, slabs and columns in service structures in which deformations in both concrete and steel were measured over varying periods of time.

Among these published accounts there is no uniformity in the use of a term for the deformation due to sustained stress, it being variously designated as "plastic flow," "time yield," "time deformation," "flow," etc., and in the majority of tests reported the factor of shrinkage has been combined with that of flow without attempt at separation, their sum being considered as the "time yield." Obviously, "flow" is primarily a function of magnitude of stress, while "shrinkage" is primarily influenced by the surrounding moisture conditions, which determine the rate at which the concrete is relieved of its uncombined moisture.

At the end appears a summary of conclusions and a bibliography of articles, papers and discussions pertaining to flow and shrinkage.

Notes on the Effect of the Time Element in Loading Reinforced Concrete Beams, by W. K. Hatt, Proc. Am. Soc. T. M., vol. 7, 1907, p. 41. The tests were made during 1905 to 1907 at the Laboratory for Testing Materials, Purdue University, under the following loading conditions:

1. Continuously increased until failure.
2. Repeated loads.
3. Sustained loads.

The beams were of 1:2:4 broken stone concrete, 8 in. wide by 10 in. to the depth of the steel. They were loaded at the one-third points, the span length being 9 ft. except for the sustained load tests for which the span varied from 8 to 12 ft. in order to obtain various desired stresses in the steel. The percentage of steel was 1.00 except for the sustained load tests, where the percentage varies from 0.75 to 1.50 for the reason stated above. Age at test was about 60 days. It appears that the beams were stored in open air, without tempera-

ture control and without temperature or humidity records, and no account is taken of shrinkage due to moisture change. Following are tabulated the important results, average values being given.

DEFLECTION OF REINFORCED CONCRETE BEAMS (HATT)
Values in Inches

Approx. Stress in Steel Lb. per Sq. In.	Ordinary Load, Testing Machine	Repeated Load			Sustained Load, 2 Mo.
		Times Applied	Deflection	Set	
3000	0.050	1 710	0.025 0.035	0.005 0.005	0.100
8000	0.080	1 500	0.070 0.155	0.010 0.055	0.160
16000	0.140	1 515	0.195 0.250	0.060 0.090	0.290
29700	0.210	1 550	0.285 0.300	0.095	0.360

INCREASE IN DEFLECTION UNDER SUSTAINED LOAD (HATT)

Initial Stress, Lb. per Sq. In.	First Recorded Deflection, In.	Deflection after 1½ Months, In.
3000	0.04	0.10
8000	0.10	0.17
16000	0.15	0.29
29700	0.20	0.36

In his discussion Hatt says, "These results, taken together, show a sort of plasticity in concrete, by which it yields under the action of a load applied for a long time, or applied a number of times."

While the conditions surrounding the tests are not sufficiently well established to make the quantitative data of much value, these tests made nearly a quarter century ago clearly demonstrated that the time yield or plastic flow of concrete under the action of sustained flexural stresses was a quantity of considerable magnitude.

Shrinkage and Time Effects in Reinforced Concrete, by F. R. McMillan, Univ. of Minn., Studies in Eng. Bull. 3, 1915.

The tests here reported were made at the University of Minnesota in the years 1913-1914 under the direction of F. R. McMillan. They comprised observations on several beams and slabs (stored in air) which were subjected to sustained load. Deformations were measured between points set in the concrete in the top of the beams or slabs and in the steel in the bottom. Center deflections were also observed. The temperature ranged between 60 deg. and 80 deg. F. No record was kept of the humidity. The object of the tests was

to study shrinkage, plastic flow (called time effect in the paper) and Poisson's Ratio.

Test of a 30-in. by 5½-in. by 12-ft. beam.—The concrete was a 1:2:5 mix, the coarse aggregate being crushed limestone. 8 by 6-in. test cylinders made from the same concrete gave at 27 days an average compressive strength of 2500 lb. per sq. in. and a modulus of elasticity at f_c = 1000 of 3,500,000 lb. per sq. in.; 4 by 5-in. transverse specimens gave an average modulus of rupture of about 500 lb. per sq. in. The steel ratio for the longitudinal was 0.00796 at a depth of 4⅝ in. from the top of the beam. The transverse steel was ⅜-in. plain round rods spaced 12 in. on centers at a depth of 5 in. No curing data are given except the statement that the beam was left exposed to the laboratory air. At the age of 41 days the beam was placed on supports thus applying a dead load calculated to produce a maximum steel stress of 8000 lbs. per sq. in., and a concrete stress of 335 lb. per sq. in. was applied. At 80 days a live load (sand-bags) of 500 lb. was applied, effective at the third points. After sundry applications and removals of load, at the age of 107 days the full live load of 1500 lb. calculated to give the same maximum stress as the dead load, was applied and sustained except for periodic instantaneous removal and reapplication.

The effect of shrinkage was determined by transverse measurements and these corrections have been applied to the longitudinal deformation given.

The actual stresses for a live load change calculated from the observed deformations, assuming a modulus of elasticity of 30,000,000 for the steel and 3,500,000 for the concrete, are about 300 lb. per sq. in. for the concrete and about 2600 lb. per sq. in. for the steel. The deformation due to stress in the concrete at the center of span after almost 60 days of dead load, was about twice the instantaneous deformation due to dead load. After 600 days, the total deformation due to stress was about three times the sum of the instantaneous dead and live load deformations. At the end of this period, the total deformation at the center of the span, due to stress, was about 0.11 per cent of the gage length. Including the shrinkage, the total deformation after 600 days was about four times the instantaneous deformation. The observed deformation in the steel at the end of a 530-day period which began with the application of the live load, which deformation corresponds to a steel stress of 12,000 lb. per sq. in., was about five times the instantaneous live load deformation. The effect of shrinkage before cracking or bond failure occurs is to decrease the observed steel deformation.

Tests of four 4 by 5 by 42-in. beams.—This group of tests was undertaken to study further the time effect and shrinkage in concrete. Four similar beams were made and stored under the same conditions for the entire period of test. On one which was without load, the shrinkage alone was measured, while on the others, the flow due to sustained load over a period of time was observed.

The concrete was 1:2:4 mix, the coarse aggregate being crushed limestone of pea size. The specimens, which were 4 by 5 by 48-in. beams, were covered with wet sacks at the time of casting, but were left open to the air after the

first day. The depth to steel was 4½-in. The steel ratio was 0.0078 for the first beam and 0.0085 for the others.

Each beam was tested on a 42-in. span with a center load of 860 lb., calculated to give approximately 650 lb. per sq. in. stress in the concrete and 15,000 lb. per sq. in. stress in the steel. Measurements of longitudinal deformation were made both on the top of the beam and in the steel, and center deflections were observed.

It appears that there was considerable variation in humidity conditions with a considerable resulting effect on the regularity of shrinkage and with some effect on flow. The total deformations due to load alone after a year were from 0.08 to 0.12 per cent, which values were from 3 to 4 times the instantaneous deformation.

Slab tests.—A 6 by 8-ft. by 3-in. thick two-way slab, supported at four edges, and a 10 by 10-ft. by 4¾-in. thick one-way slab, supported at two edges, were also tested under the action of sustained load.

For the greater part of the time, the live load on the 6 by 8-ft. slab was 100 lb. per sq. ft. Only the dead load of its own weight was applied to the 10 by 10-ft. slab.

Deformation and deflection measurements were made as in the beam tests, except that the shrinkage observations were made on the edges of the slabs over the supports.

Much the same results as were obtained in the beam tests as regards time deformation, time-deflection trends, and ratio of total load deformation to instantaneous load deformation, were obtained. Two noteworthy results manifested themselves. Due to shrinkage alone, the steel of both one-way and two-way slabs was under considerable compression, this shrinkage producing a deflection which continued to increase as the shrinkage increased; and in the two-way slab, the yielding of the concrete due to sustained stress was a higher multiple of the instantaneous load-deformation in the transverse direction than in the longitudinal direction, showing that the rate of yielding increased with stress.

Significance.—These tests served not only to demonstrate that concrete does yield considerably with time, but also point to the probable effects of flow, especially when combined with shrinkage, on compound structural members, such as reinforced concrete beams and slabs.

In tests of beams and slabs, with the zero of live load measurements established under dead load conditions of no cracks and the steel under compression due to shrinkage, if cracks are produced by the application of live load, there would be thrown on to the steel the dead load tension previously carried by the concrete and thus the apparent deformation due to live load would be greater than its true value. Also the gradual increase of steel stress with time may be due either to further yielding of the concrete in tension, thus opening up new cracks, or to a complete breaking down of the bond through the region of maximum moment.

In members subjected to flexure, the continued deflection is augmented by shrinkage, since the compression fibers, in addition to yielding plastically,

must shrink a greater amount than those on the tension side because of the restraining effect of the steel. Furthermore, the continued shrinkage on the tension side operates to increase the number of cracks and thus increase the average deformation, and hence the deflection.

In regard to the slab tests, it was found that under the condition of full dead and live load stress, the tensile steel at center of span showed compression after a year of test. It is possible that in a service structure, where a slab is not free to shrink in all directions, this condition would not occur.

The test of the 10 by 10-ft. slab showed that the time yield had a much greater effect on the continued deflection than did the shrinkage.

Thus it appears that in determining the probable total actual stress in any flexural member after any period of time, flow, coupled with shrinkage is a factor for consideration.

Time Tests of Concrete, by A. H. Fuller and C. C. More, Proc. Am. Conc. Inst., vol. 12, 1916, p. 302.

This paper reports tests made in 1915 at the University of Washington in connection with measurements of deformation in a reinforced concrete building, as result of observations that deformations in the building increased with time as the load remained constant. The test consisted of first applying load to 3 compression specimens (cast from the same concrete as, or cut from, the building), according to a schedule which was approximately the same as the loading schedule for the building, after which treatment the load was allowed to remain constant. The time from the beginning of the first part of the test to the beginning of the sustained load test, is different for each specimen. No information is given regarding the character of the concrete and the curing and environmental conditions of the building and specimens, nor is it stated whether the observations in the building were on compression or flexural members. Shrinkage was included in the values of deformation.

The thesis of the authors is stated as follows: "The modulus of elasticity is a measure of the relation of stress to deformation as determined from time tests in which the rate of application is the same (as in a structure on which time-load observations were being made) would furnish means for making a fairly satisfactory determination of the existing stresses." This would appear to be a satisfactory method of approximately determining the stresses in a structural member under observation provided only that all the influencing conditions are the same for the specimen as for the structure.

For the sustained load tests, the data which are given by the investigators represent values of deformation measured from the start of the previous loading operations and which values therefore include what flow occurred during the previous test. A comparison of the rates of flow, however, is instructive and in the following table are shown the flow between the 25th and the 60th day after the beginning of the test.

From this tabulation it will be noted that the rate of flow is greater for higher stresses than for lower stresses.

A discussion of this paper by F. R. McMillan presents data in diagramatic form illustrating the use of the above mentioned method in determining the

INCREASE IN DEFORMATION IN PER CENT
25 to 60 Days After Zero of Test

Specimen	Stress, Lb. per Sq. In.	Increment of Deformation, %
A	650	0.064
B	650	0.060
C	1150	0.090

probable actual stresses in a structure. Cases of columns in a service structure are also cited, in which it is shown that some 200 days after measurements were begun an increase in steel stress due to combined flow and shrinkage amounted in one case to as much as 22,000 lb. per sq. in.

The Flow of Concrete under Sustained Load, by E. B. Smith, Proc. Am. Conc. Inst., vol. 12, 1916, p. 317.

In 1914-1915, E. B. Smith, then with U. S. Bureau of Public Roads, Washington, D. C., conducted flow tests on a reinforced concrete beam and on concrete compression cylinders.

For the flexural tests there was employed a gravel concrete beam of 1:2:4 mix; reinforcement, approximately 0.75 per cent; dimensions 5 in. wide, 8 in. full depth, 7 in. effective depth, clear span 10 ft.; loaded at center with concentrated load of 1225 lb., calculated to produce a maximum stress of 1000 lb. per sq. in.; loaded at the age of two weeks, strain measurements along top fibers of beam. Load was sustained for 19 days, when it was released and readings were continued for 9 days more. Storage conditions are not stated.

Test results show:

1. Deformation increased with length of period of sustained load (including both shrinkage and flow).
2. A maximum deformation at end of 19 days for the top fibers of the beam at the center of span was 0.035 per cent.
3. Maximum deformation was 2½ to 3 times the instantaneous deformation.
4. Instantaneous deformation was greater than instantaneous recovery after 19 days of load.
5. Plastic recovery very small.

The compression tests were made on two 6 by 24-in. cylinders of a 1:2:4 gravel concrete loaded to a stress of 700 lb. per sq. in. One was stored in air and one was kept saturated with water. Measurements of longitudinal deformations were made on a 20-in. gage length. Corresponding unstressed cylinders were maintained under parallel storage conditions, in order that dimensional changes due to causes other than stress might be determined. All cylinders were cured for 29 days in wet sand and the dry storage specimens were thereafter kept 7 days more in air before loading.

The wet specimen was unloaded after 17 days under load, at which time the flow (making corrections for shrinkage) was 0.036 per cent. The dry specimen

was unloaded after 28 days of sustained load, the flow being 0.046 per cent. After 17 days under load, the flow in the wet specimen was about 82 per cent of that of the dry specimen.

Tests of Large Reinforced Concrete Slabs, by A. T. Goldbeck and E. B. Smith, Proc. Am. Conc. Inst., vol. 12, 1916, p. 324.

During the period 1914-16, the authors of the above named paper conducted at the U. S. Bureau of Public Roads, Washington, D. C., tests on reinforced concrete slabs, during which tests a time-yield was noticed. In part of their discussion they offer the following evidence which is not supported by any attendant data:

"During the tests of one of these slabs . . . this phenomenon was first discovered and a series of experiments was begun to obtain data regarding the flow of concrete. Since it is not the object of this paper to discuss the facts or theory . . . the following statement . . . (is given).

"When concrete is subjected to a compressive fiber stress of 700 lb., the immediate fiber deformation is only about 50 per cent of what it will be if the stress is maintained for three weeks. Within 24 hours after the application of the load, the deformation has increased an additional 20 per cent; during the first hour the deformation may show a change of 5 per cent or more. Furthermore, the recovery of the deformations, after the removal of the load is slow and not complete.

"This fact is mentioned here to show the importance of taking all the deformation readings at approximately the same time after the load is applied, and in each case a new set of deformation readings under zero load must be taken just before each load application."

The Flow of Concrete under Sustained Loads, by E. B. Smith, Proc. Am. Conc. Inst., vol. 13, 1917.

In this paper, E. B. Smith of the U. S. Bureau of Public Roads, Washington, D. C., described time tests of compression cylinders and beams, using gravel and limestone as the large aggregates. The proportions in each case were 1:2:4, and in each group there were unstressed specimens which served as controls.

In the compression tests, the cylinders were 3 in. in diameter by 24 in. long and were loaded to a stress of 840 lb. per sq. in. Cylinders were cured in the open air and sprinkled once daily for the first week. Load was applied at the age of 28 days. Deformations were measured over a 20-in. gage length. The load was released after having been sustained for 50 days. In the following table are given values of the total deformations (elastic deformation plus flow):

It is significant that the deformation of the gravel concrete was 20 to 30 per cent greater than that of the limestone concrete.

The beams were 5 in. wide by 8 in. deep, effective depth 7 in., clear span 10 ft., steel ratio approximately 0.75 per cent, central load 830 lb., Load was sustained on the gravel beam for 34 days and on the limestone beam for 48 days, after which the load was completely removed. The concrete deformation in the gravel concrete beam was about 20 per cent greater than that for the

Days of Sustained Load	Total Deformation Due to Load, % of Length	
	Gravel Concrete	Limestone Concrete
0	0.042	0.032
3	0.087	0.060
14	0.126	0.095
50	0.157	0.124
50 Load released	0.119	0.090

limestone concrete beam, although the limestone concrete beam showed a slightly greater deflection. The concrete deformations had increased about 100 per cent within the given loading periods. Steel stress due to load increased from about 11,000 lb. per sq. in. to about 18,000 lb. per sq. in. in 34 days for the gravel concrete.

Following are some of the conclusions drawn by E. B. Smith:

The law of concrete flow is asymptotic. The flow continues at a gradually decreasing rate and yet is an appreciable amount during three to four weeks. It then continues more slowly for an indefinite period, but this change is small.

The total combined effects of shrinkage and flow in compression may amount to as much as 0.2 per cent.

The effect of flow within the material itself is either to relieve the stress condition, if the construction and loading make this possible, or to gradually change the length or position of the member.

The magnitude of flow deformations vary quite largely with the kind of aggregate and the mixture.[1] The modulus of elasticity[2] of concrete is different for each mixture and aggregate. It changes and decreases in value with time and as the flow deformations increase.

In the case of a reinforced concrete beam, the effect of flow in the concrete is to lower the position of the neutral axis, thus enlarging the compressive cross-section area and relieving the unit stress value. More stress is thrown on the steel.

In the discussion that followed F. R. McMillan brought forward data to show that in columns he had investigated in building in service, steel stresses as high as from 20,000 to 36,000 lb. per sq. in. were developed, which latter stress corresponds to a deformation of about 1½ in. per 100 ft.

Design of Reinforced Concrete Slabs, Trans. Am. Soc. C. E. Discussion by F. R. McMillan, vol. 80, 1916, p. 1743.

Two groups of test results not previously discussed in this paper are briefly reported. Tests of the first group were made on four 4 by 5-in. beams with a

[1]The great effect of moisture conditions was evidently not appreciated at that time.
[2]Here is meant the ratio of unit stress to total deformation, after any given period of time.

42-in. span. All concrete was of a 1:2:4 mix. Measurements were made on an 8-in. gage length spanning the center section of the beams. Pertinent data follow; shrinkage has been eliminated from the results by means of observations on similar unstressed beams.

TIME YIELD OF REINFORCED CONCRETE BEAMS, GROUP 1 (MCMILLAN)

Beam No.	Coarse Aggregate	Reinforcement	Calculated Stress, ($n = 15$) Lb. per Sq. In.	Age When Load Applied
11	pea size limestone	1⅜" sq. twist	670	33 days
13	pea size limestone	2 $\frac{5}{16}$ pl. round	650	33 days
118	¼" to ¾" limestone	1⅜ pl. round	730	103 days
121	sandstone	1⅜ pl. round	730	103 days

Beam	Initial Unit Deformation, Inches	Relation of Total Deformation to Initial Deformation			
		30 Days	90 Days	200 Days	530 Days
11	0.000335	2.49	2.66	3.50	4.09
13	0.000302	2.48	2.78	3.28	3.74
118	0.000246	2.40	2.84	3.29	....
121	0.000180	1.57	2.08	2.68	....

In the second group were beams and slabs with dimensions and loading schedules shown below.

Specimen	Mix and Coarse Aggregate	Time Kept Wet, Days	Depth, In.	Width, In.	Effect. Depth, In.	Span In.	Steel Ratio	Age at Dead Load Application Days
A Beam	1:2:4 limestone pea size	1	5.00	4	4.50	42	0.0085	33
B Beam	1:2:4 limestone ¼" to ¾"	not wet	5.50	30	4.62	144	0.0080	41
C Slab	1:2:4 limestone ¼" to ¾"	31	4.87	120	4.35	120	0.0046	135
D Slab	1:2.50:4 limestone pea size	57	3.00	96	2.62	72	0.0049	59

Important live load variations:
Beam A. Dead load and 860 lb. at center applied at age of 33 days.
Beam B. Loaded at ⅓ points, 500 lb. at 80 days, 1500 lb. at 88 days.
Slab C. 50 lb. per sq. ft. at 174 days after dead load application.
100 lb. per sq. ft. at 275 days after dead load application.
Slab D. 50 lb. per sq. ft. at 68 days.
100 lb. per sq. ft. at 71 days.
Live load removed at 82 days.
Live load reapplied at 94 days.

The results of tests on beams and slabs of the second group are shown in the following table.

TIME YIELD OF BEAMS AND SLABS, GROUP 2 (MCMILLAN)

Type Loading	Unit Instantaneous Deformation			
	Specimen A	Specimen B	Specimen C	Specimen D
Dead load	....	0.000200	0.000076	....
Live load				
0 days	0.000280	....	....	0.000042
3 days	....	....	....	0.000065
46 days	....	0.000035	....	....
54 days	....	0.000075	....	....
174 days	....	....	0.000058	.....
275 days	....	....	0.000043	....

Type Loading	Ratio, Total to Sum, Instantaneous Deformations			
	Specimen A	Specimen B	Specimen C	Specimen D
Dead load				
50 days	2.57	....	2.90	2.24
100 days	2.68	2.32	....	2.66
200 days	3.39	3.11	3.20	3.08
650 days	4.25	3.63	4.00	4.11
950 days	4.60	3.71	....	4.39

Consulting the results of the two groups of tests it is seen that the total deformation after a six months of sustained load is roughly three times the initial deformation, and after two years is roughly four times the initial deformation. From another point of view, the ratio of moduli for steel and concrete, n, which ranged from 9 to 15 at the time of initial loading, may increase to 20 or 30 in a few months, and from 35 to 60 in 2 years.

It is noteworthy that here for the first time were reported tests extending over a period of two years or more, and the discovery was made that flow was still taking place at the end of these extended periods of observation.

Extensometer Measurements in a Reinforced Concrete Building Over a Period of One Year, by A. R. Lord, Proc. Am. Conc. Inst., vol. 13, 1917, p. 45.

In this paper A. R. Lord reports the results of tests made in 1914 and 1915 on a 25 by 26-ft. flat slab in a reinforced concrete warehouse located in Chicago. The load, which was twice the design load, was left in place for about a year, and strain-gage measurements were taken periodically during this time. The slab was 10 in. thick and the concrete was a 1:2:4 mix. What were believed to be corrections were made in the results for temperature changes (based probably on air temperature and the coefficient for steel) but the deformation reported include shrinkage. The test load was a uniformly distributed layer of sand.

The results are best stated in the form of general conclusions, as follows: The ratio of deformations at 1 and 55 days is about 1:2.5 and the ratio of de-

formations at 1 and 230 days is about 1:5 (results not dissimilar to those of previous tests).

The continued deformation of the concrete was noted, but the rate of deformation of the steel was not so great as that of the concrete, showing that although flow took place continuously, shrinkage operated to reduce the tensile stress observed in the steel (see McMillan's paper on Time Tests of Concrete).

Due to flow, and therefore increased deflection in regions of negative moment, a shifting of the moment took place, resulting in a still greater increase in deflection at regions of positive moment.

The increase in total moment figured from steel stress was relatively small.

The "straight line theory" of reinforced concrete is inapplicable to the conditions surrounding this test even with the low stresses present in the concrete and steel.

Plasticity and Temperature Deformations, by S. C. Hollister, Proc. Am. Conc. Inst., vol. 15, 1919, p. 127.

Load deformation data are presented for tests made at Purdue University on a narrow reinforced concrete slab. Under the conditions of the test, which are not reported, the elastic deformation is about the same as the "plastic" deformation for all magnitudes of load, for both steel and concrete. Plastic deformations evidently include shrinkage. Plasticity is defined as that deformation which does not disappear upon the removal of load, i. e., permanent set.

A Study of Column Test Data, by F. R. McMillan, Proc. Am. Conc. Inst., vol. 17, 1921, p. 150, and Discussion by M. B. Lagaard, loc. cit. p. 172.

Incidental to a general discussion of reinforced concrete spiral-wound columns, McMillan reviews briefly his own and other flow data.

In his discussion of McMillan's paper, Lagaard presents more recent data regarding the column flow tests on a building at the University of Minnesota (which tests have been previously mentioned in this paper in connection with McMillan's discussion of other papers). Also there are given the results of tests on three columns in reinforced concrete warehouses.

The columns in the building at the University of Minnesota were still decreasing in length (due to flow and shrinkage) after a period of six years. The tests on these columns were begun after they had been in existence for about two months, so that observed deformations do not include those due to shrinkage and to flow for that preliminary period. Assuming a constant modulus in the steel of 30,000,000 lb. per sq. in., the increase in stress for the six years varied among the several columns from 36,000 to 45,000 lb. per sq. in. The percentage of vertical steel in these columns varied from 0.7 to 2.1.

In one warehouse, in which the columns had 3.7 per cent of vertical steel and 1.6 per cent spiral hooping and the tests were begun at an age of three weeks, the increase in vertical steel stresses for an observed period of 4½ years was 14,000 to 15,000 lb. per sq. in. It is also interesting to note that the lateral shrinkage during this period corresponded to steel stresses of from 11,000 to 15,000 lb. per sq. in. Another noteworthy feature of these tests is that in one column the vertical gage lines in the concrete showed about 20 per cent more

deformation than did those in the steel. This may have been due to the fact that the concrete and steel gage lines were not in the same vertical element and some bending was taking place.

Tests on another warehouse column containing 4.5 per cent vertical steel and 1.0 per cent spiral, showed, after nearly 5 years, a concrete deformation of 0.046 per cent and a steel deformation of 0.029 per cent, the latter corresponding to a stress of about 9,000 lb. per sq. in. In this column the vertical elements in which the concrete gage lines were located made angles of about 60 deg. and 120 deg. with the element containing the steel gage lines.

Conclusions stated by Lagaard are: High compressive stresses are likely to exist in the vertical steel reinforcement of concrete columns due to the combined effects of load, shrinkage, and time-yield.

The transverse steel will also have high compressive stress due to the action of shrinkage.

The compressive stresses in the vertical steel will be lessened when the percentage is increased.

Fatigue of Concrete, Discussion by H. F. Clemmer, Proc. A. S. T. M., vol. 23, 1923, p. 339.

In connection with what he calls "static load fatigue," Clemmer reports the following of cantilever beams subjected to static loads which produced stresses of various percentages of their moduli of rupture. "The critical percentage of stress which caused failure was found to be 71 per cent. A number of beams failed at the end of approximately 105 hours under a load causing a stress of 71 per cent of the breaking stress. With a stress of 84 per cent, the beams failed at the end of 14 hours." Clemmer also states that most of the set due to a static load is temporary and is recovered after a time.

Plastic Yield, Shrinkage and Other Problems of Concrete and Their Effect on Design, by O. Faber, Min. of Proc. Inst. C. E. (Great Britain), vol. 225, 1927-28, Part I.

Faber reports the results of laboratory tests made in 1926-27 on six simply supported beams, each 2 in. wide, 5 in. deep, effective depth $4\frac{1}{2}$ in., of span 15 ft. The concrete was approximately a $1:1\frac{3}{4}:3\frac{1}{2}$ mix with a water cement ratio of 0.69. The tension steel consisted of two $\frac{3}{16}$-in. round rods in all of the beams, with the addition of a like amount of compression steel in beams 5 and 6. As tested the dead load of each between supports was 150 lb. At 28 days test cubes of the concrete of beams 1 to 4 exhibited a compressive strength of 3400 lb. per sq. in., test cubes for beams 5 and 6 gave 3300 lb. per sq. in. and test cylinders for beams 5 and 6 gave 2600 lb. per sq. in. The steel for beams 1 to 4 had a yield point of 50,300 lb. per sq. in. and that of beams 5 and 6, 28,600 lb. per sq. in. Storage conditions as regards temperature and humidity are not stated.

All beams were stored in air continuously, except No. 4 which was kept wet after the age of 30 weeks. Beams 1 to 4 were placed upon their end supports and loaded at age of 4 weeks; beams 5 and 6, at the age of 5 weeks. Beam 3 was unloaded at the age of 30 weeks. Deflection measurements were made periodically beginning with the time of loading. Deformations in steel and

concrete were observed on beams 3 and 4 between the ages of 10 and 18 weeks and on beam 6 between the ages of 6 and 15 weeks.

The following table gives the centrally applied live loads for the several beams, and also gives calculated values of concrete and steel stress, as well as calculated and observed deflections. The calculated stresses and deflections are based upon an assumed straight line variation of compressive stress in concrete with no tension in concrete, and upon modulus of elasticity of 4,000,000 lb. per sq. in., $n = 7.5$. The observed deflections are those that took place at the instant the loading tests were begun. It will be noted that the observed instantaneous deflections are roughly 0.6 of the corresponding calculated values.

BEAM LOADINGS, STRESSES AND DEFLECTIONS (FABER)

	Beam No.					
	1	2	3	4	5	6
Live load, lb.	0	30	60	90	0	30
Calculated Stresses, lb. per sq. in.						
Concrete	676	946	1216	1487	608	851
Tension steel	13750	19250	24750	30250	13640	19100
Comp. steel	....	....	....	....	2540	3550
Calculated Instant. Deflection, in.	0.474	0.622	0.770	0.918	0.47	0.61
Observed Instant. Deflection, in.	0.30	0.35	0.40	0.50	0.29	0.37

The behavior of the beams under sustained load is indicated by the two following tables, taken from graphs prepared by Faber, the first giving observed deflections and the second giving deformations of top and bottom fibers at various ages. The values show the combined effect of plastic yield and shrinkage. Ages are counted from the day the specimens were cast. Consulting the tables, it is seen that both deflections and compressive deformations (top) increase substantially as time goes on. Further, for the tensile fibers (bottom) of some of the beams, the effect of shrinkage nearly offsets that of plastic yield.

BEAMS UNDER SUSTAINED LOADS (FABER)

Observed Deflections, Inches

Age, Weeks	Beam Number					
	1	2	3	4	5	6
4	0.30	0.35	0.40	0.50	....	....
5	0.34	0.41	0.54	0.65	0.29	0.37
10	0.57	0.67	0.89	1.01	0.55	0.67
20	0.78	0.89	1.16	1.25	0.69	0.80
25	0.80	0.93	1.19	1.26	0.71	0.82
30	0.81	0.98	1.22	1.31	....	....
40	0.85	1.01	....	1.40	....	....

BEAMS UNDER SUSTAINED LOADS (FABER)

Observed Deformations, Top and Bottom Fibers
(Positive Signs Indicate Elongations)

Age, Weeks	Observed Deformations, Per Cent of Gage Length					
	Beam 2		Beam 3		Beam 6	
	Top	Bottom	Top	Bottom	Top	Bottom
6	....	....	....	....	0	0
8	....	....	....	....	−0.0081	+0.0050
10	0	0	0	0	−0.0133	+0.0062
12	−0.0125	+0.0005	−0.0130	+0.0026	−0.0187	+0.0075
14	−0.0145	+0.0035	−0.0168	+0.0053	−0.0240	+0.0100
15	−0.0150	+0.0052	−0.0171	+0.0070	−0.0250	+0.0105
18	−0.0220	+0.0040	−0.0240	+0.0074	....	....

Assuming the deflections to vary directly with the deformations, Faber has calculated the probable deformations of the top fibers of four of the beams up to the age of 32 weeks, due to combined shrinkage and plastic yield as well as the corresponding strains which took place when load was applied at the age of four weeks. These values are compared in the following table. It is instructive to observe that the calculated deformation at the end of 32 weeks due to plastic yield plus shrinkage is on the average nearly three times the calculated deformation taking place at the time the load was applied.

COMPUTED TOP FIBER DEFORMATIONS IN BEAMS

Between Ages 4 and 32 Weeks (Faber)

Beam No.	Calculated Elastic Strain, In. Per In.	Top Deformation Due to Shrinkage and Plastic Yield, In. Per In.
1	0.00017	0.00052
2	0.00024	0.00066
3	0.00030	0.00084
4	0.00037	0.00087

From a study of these tests and other data Faber separates the element of shrinkage from plaster yield and draws the following conclusions:

1. At a stress of 1000 lb. per sq. in., the plastic yield in 36 weeks is approximately equal to the total shrinkage and each is approximately twice the elastic strain.

2. The plastic yield varies with the elastic strain, being roughly double its value, while the shrinkage remains constant.

3. The total deformation in 36 weeks is roughly 6 times the elastic strain at 600 lb. per sq. in., about 5 times that at 1000 lb. per sq. in., and about 3½ times that at 1400 lb. per sq. in.

4. In beam No. 6, with compressive reinforcement, for the period 5 to 22 weeks, the calculated increase in stress due to shrinkage and plastic flow was 11,700 lb. per sq. in. for the compression steel, and 3,800 lb. per sq. in. for the tension steel. For beams 2 and 3, the calculated additional stresses in the steel from 4 to 36 weeks were 1,400 and 4,300 lb. per sq. in. respectively.

Conclusions

A review of the abstracts of the preceding pages giving the results of tests by various investigators justifies the following general conclusions:

1. All concrete under the action of sustained stress flows or plastically deforms for a long period of time, certainly for several years. This is true regardless of whether the concrete be plain or reinforced and is true for beams and slabs subjected to bending moment as well as to columns subjected to direct compression.

2. Regardless of the character of the concrete or of the conditions surrounding it, flow increases with the magnitude of the sustained stress, as well as with time. Other things being equal the flow is proportional to the instantaneous deformation or that which takes place as the load is applied.

3. When concrete is in air, shrinkage continues for a long period of time, the amount of shrinkage depending upon the dryness of the atmosphere, but being independent of stress conditions. This is true for reinforced as well as for plain concrete.

4. In a reinforced concrete column the combined effect of shrinkage and flow under sustained load is gradually to relieve the compressive stress in the concrete until the yield point of the steel is reached. There is evidence that under normal load conditions the spiral hooping of a column surrounded by dry air, due to lateral shrinkage, is always in compression.

5. In beams and slabs reinforced in the normal manner the combined effect of shrinkage and flow is gradually to lower the neutral axis at sections of positive moment and to raise the neutral axis at sections of negative moment. This results in an increase in deflection, in a reduction of compressive stress upon the extreme fiber of the concrete, and in an increase in the tensile stress in the steel. In continuous and restrained beams and slabs there is also brought about, due to flow, a gradual redistribution of moments tending to equalize those of opposite sign.

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Basis of Design For Hurricane Exposure

Report of Committee 308

BY ALBERT SMITH, AUTHOR CHAIRMAN*

This brief report prepared by Mr. Smith has been submitted to the critic members of his committee: D. C. Coyle, Hardy Cross, W. J. Knight, G. A. Maney, L. E. Ritter and W. J. Spiker and since redrafted. The report is here supplemented by the Author-chairman's discussion and discussion invited for subsequent publication, from the critic members of the committee and others.—EDITOR.

THE STRUCTURAL frame shall be designed to resist all the force of the wind on the total exposed surface.

In computing wind loads, the following unit pressures, varying with the height above sidewalk shall be used:

0 ft. to 200 ft.—20 lb.
200 ft. to 300 ft.—25 lb.
300 ft. to 400 ft.—30 lb.
400 ft. to 500 ft.—35 lb.
500 ft. —40 lb.

except that structures located completely within a solidly built up section may be designed for 10 lb. of pressure below the highest complete shelter found within 400 ft. of its walls.

Spread footings, pile footings and caps and caissons shall be so designed that the loads and stresses from the combination of dead, live, and wind loads shall not exceed by more than 50 per cent the design units for dead and live alone.

All columns whose vertical wind load does not exceed 20 per cent of the dead and live load may deliver what wind moment they carry in their lowest section into square spread footings,

*Smith and Brown, Engineers, Inc., Chicago.

square pile clusters and to caissons without increasing such footings for wind. Where the footings are rectangular they shall be examined for soil pressure, pile load and steel and concrete stresses and if the increase from wind vertical and moment exceeds 50 per cent of the dead and live units they shall be redesigned.

All members of the structural frame whose connections cause them to bend when the building deflects under wind may be regarded as sharing in the resistance to the wind.

The structural frame shall be regarded as a unit in resisting the wind pressure by the bending of its members and the moments in all joints of the structure shall be assigned in proportion to the stiffness of the joint which depends on the stiffness of the members connected at such joints. Where two members share the moment at a given joint, the moment shall be divided between them in proportion to their respective stiffness.

The moments so found in the members may be reduced to the face of the column, for beams, and to the top and bottom of the beams, for columns. The connections of all members shall be designed for these reduced moments by the stress units hereinafter specified.

The stiffness of the joints of braced bents at floors where the bracing begins or is stopped, is less than that of joints at the same floor of continuously braced bents whose members have the same properties. At offsets and at all places where shallow or no beams are placed between columns which have stiff girders in floors above, and where downward going bracing bays start, an uncertainty exists as to the amount and location of the horizontal wind shear which will be transmitted.

This horizontal wind shear shall be assumed to be transmitted at three floors and in assigning stresses to girders, columns, and horizontal floor bracing, an excess of at least 75 per cent of the detoured wind shear must be divided among these three floors to cover this uncertainty.

Where the bracing for a tower apparently could be stopped at the main roof line and transferred to other lines the tower bracing shall be carried at least two stories below the top of the bracing of the enlarged section below.

Where a column of a braced bent is carried on a girder, allowance shall be made in distributing the stresses, for the deflection of the girder, as affecting both the moment and the vertical load of the riding column and the columns adjacent in the bent.

If columns designed for dead and live loads are found to be inadequate for combined stresses of dead, live and wind loads, they shall be redesigned to make them adequate by increasing the size, or adding to the vertical steel or by increasing the spiral or by adding vertical steel wired to the spiral at the corners of the column or by a combination of these devices.

Where the spandrel beams are relatively shallow, the wind stresses in interior flush beams and joists will be relatively large. Such flush beams and joists shall be redesigned for the combined stresses from dead, live and wind loads.

DESIGN UNITS

Beams—Where no wind rods are used, the capacity of beams to resist wind moment shall be computed for a fibre stress of 500 lb. per sq. in. in the bottom fibres of the beam, neglecting the steel provided for static loads. Where the moments exceed the capacity of the concrete, steel wind rods shall be added of such cross sectional area that, taking the distance between the centers of the top and bottom rods as the effective depth the fibre stress shall not exceed 27,000 lb. per sq. in. for medium grade steel, neglecting the resistance of the concrete.

The bond of these wind rods shall not exceed 150 per cent of the bond unit for static loads, except that where a length curved on a radius of not less than 10 in. is inserted in the column the bond of such curved length may be 225 per cent of the bond unit for static loads. Where straight wind rods project through the column into the beams, the bond in the column must be equal to the sum of the compression in the steel on one side and the tension on the other. Wind rods shall project into the beam to the point at which the combined positive static and positive wind moment

will not stress the positive static steel more than 150 per cent of the static steel unit stress.

For the combined static and wind shear the unit of diagonal tension and stirrup stress shall be taken as 150 per cent of the units for static load alone. Girders, whose ends are connected to columns by wind rods, shall be counted as fulfilling the condition of "Special Anchorage" for static load.

Columns—The combined stress, consisting of dead and live load, vertical wind load, and the wind moment transformed into equivalent centric load, shall not exceed 150 per cent of the capacity by code for dead and live loads only. The wind moment in the column at the top or bottom of the beam may be transformed by the formula Equiv. Cent. Load $P_{WM} = \frac{AMv'}{I}$, where A is the area inside the spiral, v' the distance of the spiral from the center of gravity and I the moment of inertia of the total section of the column.

DISCUSSION BY ALBERT SMITH, AUTHOR-CHAIRMAN

WIND PRESSURE UNITS

TO ESTIMATE the expectable velocity of wind which may produce the maximum pressure on a building, two rather indefinite factors must be applied to the records of maximum velocity by three-cup anemometers or four-cup anemometers corrected to three-cup equivalents—one for height of instrument, and one for gust intensity. See Table 1.

The reported velocities are average velocities for five minutes. In that time the record of pressures shows that the velocities have varied from 25 per cent of the average to 175 per cent of the average. On the face of it, we should add 50 per cent to the corrected reports of average readings to determine the proper design pressure. These gusts, however, may be of small cross section.

It is probably sufficiently proved that near the ground the width of gusts is small. It is not proved that at a height of say 150 ft. in a clear air way the width of gusts is small. Fleming[1] quotes Stanton's conclusions from observations taken on the high level footway over the draw-span of the Tower Bridge at

[1]Wind Stresses in Buildings, p. 53.

TABLE 1—MAXIMUM REPORTED WIND VELOCITIES, DEPARTMENT OF AGRICULTURE—BULLETIN W-1926, VOL. 1, 2, 3.

Years During	City	4 Cup	Corr. 3 Cup	50 Per Cent Incr.	Elev. Station
47	Albany	70	57	86	115
48	Augusta, Ga.	62	50	75	77
50	Boston	72	58	87	188
32	Cairo	84	67	101	93
50	Charleston	106	85	128	92
42	Charlotte, N. C.	72	58	87	62
49	Cheyenne	78	63	95	101
48	Chicago	84	67	101	131
52	Cincinnati	54	44	66	51
51	Cleveland	73	59	89	201
44	Columbus	78	63	95	222
7	Dallas	62	51	77	227
49	Davenport	72	58	87	79
10	Dayton	62	51	77	173
48	Denver	75	60	90	113
51	Detroit	87	70	105	258
49	Duluth	78	63	95	47
47	Eastport	78	63	95	85
49	Erie	71	57	86	166
25	Evansville	77	62	93	116
41	Fort Smith	74	60	90	94
22	Ft. Worth	66	54	81	114
	Grand Rapids	66	54	81	87
42	Helena, Mont.	70	57	86	112
51	Indianapolis	71	57	86	230
49	Louisville	74	60	90	234
48	Memphis	75	60	90	97
25	Minneapolis	84	67	101	208
46	New Orleans	86	69	104	84
37	New York	96	77	115	454
30	Philadelphia	75	60	90	182–367
49	Pittsburgh	70	57	86	410
49	Portland, Me.	61	50	75	106
51	Portland	55	45	68	106
16	Providence	82	66	99	251
34	Pueblo	64	52	78	86
48	St. Louis	80	64	96	303
32	St. Paul	102	81	122	261
52	San Francisco	64	52	78	243
44	Sandusky	69	56	84	67
50	Savannah	88	71	107	194
29	Seattle	64	52	78	250
19	Syracuse	66	54	81	113–79
49	Toledo	84	67	101	...
41	Vicksburg	62	51	77	73
36	Walla Walla	65	53	80	65

London. Stanton declared that he found that gusts might occur whose intensity would be practically uniform over the whole structure. Where a comparatively steady high velocity wind is

blowing between 1000 ft. and 2000 ft. above the earth, the gusts of which we have record originate by the deflection of masses of air from the swiftly moving stream toward the earth. It is not reasonable to suppose that these deflected masses are splinters of air current less than 100 ft. wide. On the contrary, it is logical to assume that these are very much larger masses, which as they approach the earth are split up successively by the roughness of the surface of approach and the infiltration of the slower currents below. The maximum velocity at the center of the gust is probably indicative of the velocity of the main air current above. Thus a one minute velocity in New York of 91 mi. per hr. at 454 ft. elevation shows a 30 second velocity much greater, and is ample justification for assuming that the main current of air is traveling much faster than 100 mi. per hr.

Pressure is determined by applying a coefficient to the square of the velocity. Dryden and Hill found at the Bureau of Standards[2] that a tower model whose proportions were 1:1:3 received a total pressure which can be expressed as $P = .00384\ V^2$, and they predict that higher towers will show a larger coefficient. Other observers have found that long plates receive a higher unit force than square plates.

It is my contention that we should start with 40 lb. (100^2 x .004) for say 1500 ft., and not diminish it until we reach the elevation at which we are sure that the width of the gust having an average pressure of 40 lb. is less than the width of the structures to which we propose to apply the unit. I believe that this point cannot safely be taken as higher than the 500 ft. point above sidewalk, or 300 ft. above the base of the clear air-way

I propose that the pressure units should be 40 lb. above 500 ft. from sidewalk, 35 lb. between 400 ft. and 500 ft., 30 lb. between 300 ft. and 400 ft., 25 lb. between 200 ft. and 300 ft., 20 lb. between 100 ft. and 200 ft., and 15 lb. below 100 ft.

I have no confidence that there is any place in the country where these pressure units will not be exceeded during the next 20 years, but I am confident that in buildings carefully designed for the stress units set forth by the committee and for these pressures, there will be no overstrain that will damage the structural frame.

[2]Scientific Paper No. 523, p. 731.

COMPUTATION OF WIND STRESSES IN BUILDINGS

In a building of 22 stories, having 45 columns, the wind causes stresses whose total number for two directions of wind is nearly 10,000. A large part of these stresses are not important to the designer. A few upper stories may be dismissed from consideration for all stresses. Most intermediate columns have minor axial stresses from wind which need be computed only for one or two stories. With all these deductions the number of stresses to be computed and marked on a set of framing diagrams is very large. Any method which will give us, reliably, the wind stresses in a building which will not be exceeded by 5 per cent nor be in excess more than 10 per cent should be quite satisfactory to the designer.

Several short methods of calculation for single bents have been proposed. Some appear likely to have a maximum error of only 5 per cent. If however, the amount of wind which will come in this bent is not determined within 25 or 30 per cent the accuracy of distribution within the bent is of little use.[3]

Any structure must be regarded as a whole in computing its wind stresses, and when the building is unsymmetrical in any respect, and is of great height, the labor of making the closest practicable approximation to the true stresses is not only justified, it is essential.

For a symmetrical building with all bays in the same direction equal, and having intermediate columns so turned that their stiffnesses are proportionate in the bents in which they are intermediate, a fair approximation of the wind moments at the joints may be made by the following assumptions.

a. All points of contraflexure at the middle of the lengths of all columns and beams.

b. Equal vertical wind shears in all beams.

Each floor of a building deflects horizontally as a unit under wind pressure. If the building is symmetrical, both for layout

[3]Mr. Fleming in his very interesting book, "Wind Stresses in Buildings," (John Wiley, 1930) quotes Prof. Burr's statement that "So long as the stresses found by one legitimate method of analysis are provided for, the stability of the structure is assured." We should not allow this truth, so excellently stated, to make us satisfied with crude methods of finding wind stresses in buildings. After all, are they "legitimate?" I take it Prof. Burr did not mean that if the stresses as arbitrarily divided between 40 or more paths are provided for, the structure is safe. Nor if a weight be hung on a thread and a rubber band of the same length, that the factor of safety is the weight divided by the sum of the two ultimate strengths. The designer is charged with the duty not only of supplying enough strength, but also of putting it in the right place.

and framing, all points along the wind exposure of the floor deflect equally.

Even in the most symmetrical building, however, computations made with the assumptions given above must have some approximate corrections. All buildings have, in one or the other direction of the wind, much shallow framing whose resistance to the wind, if it is symmetrical, may be subtracted directly from the total moment at the given floor.

Even in the most symmetrical buildings the I of the end columns of the principal bents may be so much less or so much more than one half that of the intermediate columns of these bents, that the calculations by the method outlined above will be seriously in error.

Either the columns should be adjusted for stiffness or an allowance should be so made in the stresses that the designer can be confident that his assumed stresses will not be exceeded.

Where a building is unsymmetrical but has equal column spacing, the eccentricity of the wind load with respect to the stiffness of the building can be computed and adequate allowance made in the stresses without too greatly loading the design.

When, however, for a given direction of wind, the beam spans vary widely or the wind resistance is very unsymmetrical or the beam and column stiffnesses are made disproportionate to their wind function by other load requirements, or as often happens, all of these unfavorable conditions exist, we must use some more powerful method of computing the distribution of the wind stress among all the resisting members with a reasonable degree of accuracy.

The method shown herewith has been developed in tabular form so that engineering draftsmen might fill in the tables and compute the stresses and learn to understand the method while they computed. For a building with 45 columns we estimate that the tabulation of $\frac{1}{R'}$, the true K, and the distributed moments for one floor can be made by one man in two and one half days. The total work for three floors, in two directions and the placing of computed and interpolated stresses on the general stress diagram is of the order of two men for two weeks.

There will still remain the examination of columns for wind stress, and in some cases the correction of many of them, and the design of the brackets for a steel frame, or the wind rods for a concrete building. A very competent designer, with a draftsman helper, would probably find plenty to do for a week after all stresses are known.

The wind stresses in Eddystone Belmont were first laid out by the approximate method, roughly corrected for eccentricity of column spacing and column and beam stiffness. These figures were used in the design for estimate, and contract. As always, after the estimate plans go out there was plenty of time for computing more exactly. We found, by calculation, that in the major stresses our allowances were ample, but that we had in some bents used an excess of steel, and carried the wind reinforcing too high, and in some bents we found our wind reinforcing deficient. A careful approximation of the stresses was more rapid than the calculation, but even though our allowance for error was large, we needed the reassurance of the calculation. For this building the calculations here shown might have been greatly shortened. The calculations for joists and for flush beams might have been lumped together in an approximate total without danger of a large error in the final moments in the important beams. These calculations after having been originally made with some abbreviations were expanded to a complete table to enable readers to study the relation between moments of joints with varying conditions.

AN EXAMPLE OF WIND STRESS COMPUTATION

To apply, with reasonable amounts of labor, the equation shown in Fig. 1, it has been found necessary to develop charts from which the factors $\frac{1}{I_c}$ and $\frac{I_B}{L_B}$ can readily be obtained.

I have made, but not given in this paper, the tabulation of the calculation of the I's of square columns varying from 11 in. to 33 in. in gross dimension and with spirals varying from 8 in. to 30 in. For these calculations the E for 3000-lb. concrete was taken as 3,600,000 and the n as 8 and for 2000-lb. concrete, as 3,000,000 and 9.6. The values of I are computed as:

$$I_{2000} = .0833D^4 + .008p\ (D - 3)^4$$
$$I_{3000} = .10D^4 + .008p\ (D - 3)^4$$

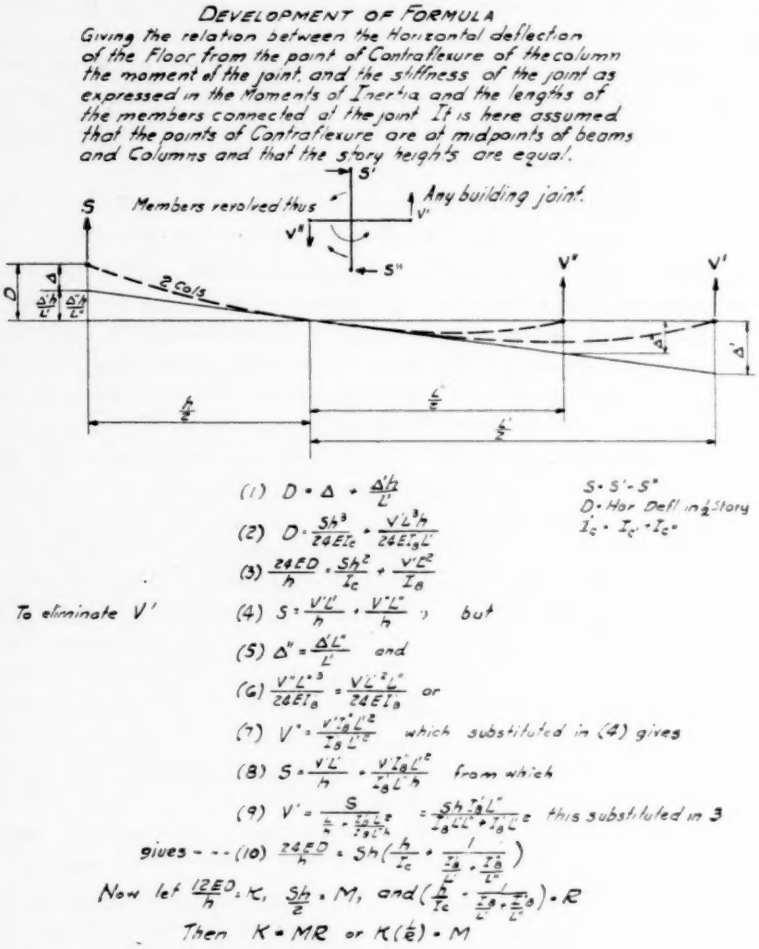

DEVELOPMENT OF FORMULA
Giving the relation between the Horizontal deflection of the Floor from the point of Contraflexure of the column the moment of the joint and the stiffness of the joint as expressed in the Moments of Inertia and the lengths of the members connected at the joint It is here assumed that the points of Contraflexure are at midpoints of beams and Columns and that the story heights are equal.
Members resolved thus
Any building joint.
S
S'
S"
V'
V"
V'
Σ Col's
D
Δ
Δ'h/L'
Δ'h/L"
Δ'
Δ"
h/2
L'/2
L'/2
(1) D = Δ + Δ'h/L'
(2) D = Sh³/24EI_c + V'L'³h/24EI_B L'
(3) 24ED/h = Sh²/I_c + V'L'²/I_B
To eliminate V'
(4) S = V'L'/h + V"L"/h ; but
(5) Δ" = Δ'L"/L' and
(6) V"L"³/24EI_B = V'L'²L"/24EI_B or
(7) V" = V'I_B L'²/I_B L"² which substituted in (4) gives
(8) S = V'L'/h + V'I_B L'²/I_B L"h from which
(9) V' = S/(1/h + I_B L'²/I_B L"h) = Sh I_B L"/I_B L'L" + I_B L' this substituted in 3
gives - - - (10) 24ED/h = Sh(h/I_c + 1/(I_B/L' + I_B/L"))
Now let 12ED/h = K, Sh/2 = M, and (h/I_c + 1/(I_B/L' + I_B/L")) = R
Then K = MR or K(1/R) = M
S = S' - S"
D = Hor Defl in ½ story
I_c = I_c' + I_c"

FIG. 1—DEVELOPMENT OF FORMULA

It is obvious that for the smaller values of p, the diameter of the steel ring is larger and the I of the steel proportionately greater than for the larger values of p, also that for large spirals the I of steel is greater than for the same percentages in smaller columns. The $.008p$ used here is a reasonable mean of these variations for the larger percentages. For the smaller percentages, the steel contribution to the I is small. In each square on this table, in addition

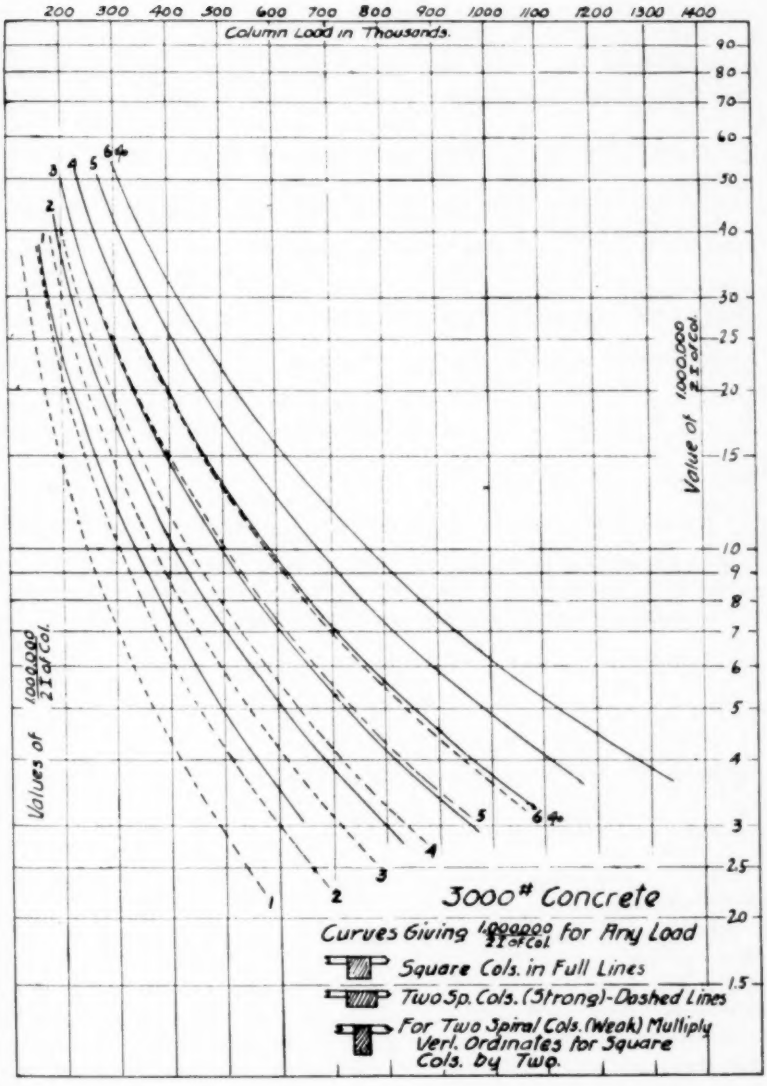

200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400
Column Load in Thousands.
90 80 70 60 50 40 30 25 20 15 10 9 8 7 6 5 4 3 2.5 2.0 1.5
Value of 1000000 / Σ I of Col
Values of 1000000 / Σ I of Col.
3 4 5 6 4
2
1 2 3 4 5 6 4
3000# Concrete
Curves Giving 1000000 / Σ I of Col. for Any Load
Square Cols. in Full Lines
Two Sp. Cols. (Strong)-Dashed Lines
For Two Spiral Cols. (Weak) Multiply Verl. Ordinates for Square Cols. by Two.

FIG. 2

to the I in thousands, there are shown the maximum column load in thousands and 1000 times the reciprocal of $2I$. In Fig. 2 are shown the curves plotting 1,000,000 times these reciprocals against the maximum safe column load. Numbers obtained from these curves are to be multiplied by the story height in feet.

The use of the load to determine the reciprocal of the stiffness factor of columns leads to error because the curves are laid out for the minimum section which can be used to carry that load. If an excess of steel is used in any column, the $\frac{1{,}000{,}000}{2I}$ as determined by the actual load and the excessive percentage will be too large, and the stiffness factor will be too small. The effect of this error on the final moments is small but some effort has been made in using these curves to make the figures for $\frac{1{,}000{,}000}{2I}$ relative by roughly adjusting them to each other after the columns on Tables 2 and 3 have been filled in. The joints, whose final stiffness factors are most affected by these errors, are those whose moment is small and the most important stresses are very slightly affected.

Fig. 3 gives the beam curves from which $\frac{I}{1000L}$ can be obtained for spans ranging from 8 ft. to 40 ft., and for depths varying 9 in. to 48 in. The scale on the left edge permits the direct determination of the reciprocal for joints having only one beam. The curves are made for beams 8 in. wide and having only one flange. The I of the stem—without steel—is counted as $\frac{8d^3}{12}$, and the I of the one flange, with the steel, is counted as also equal to $\frac{8d^3}{12}$. For wider beams I have added $\frac{d^3}{12}$ to the I for each inch of increased width of stem, and for beams with two flanges I have added $\frac{8d^3}{12}$.

A table of coefficients is given for increase of stem widths for one and two flange beams.

It is, of course, quite impossible to make a rational determination of the I of concrete beams for stiffness. It is quite clear that the method here used is only approximate. Short spandrel beams, for instance, are likely to have very light reinforcement for dead and live loads. The final error from this cause is much reduced by the fact that the short beams have more wind moment than the long beams, and will certainly have as much wind reinforcing. Being short, their wind reinforcement will cover so large a portion of their length that the final error in the I is not so large as at first appears.

These assumed values for the I of beams tend to give us excessive stiffness for short beams. The practice here followed of using c. to c. of columns minus one quarter of the width of each column as the span of the beam tends to error in the other direction for short beams. The resultant of these errors is

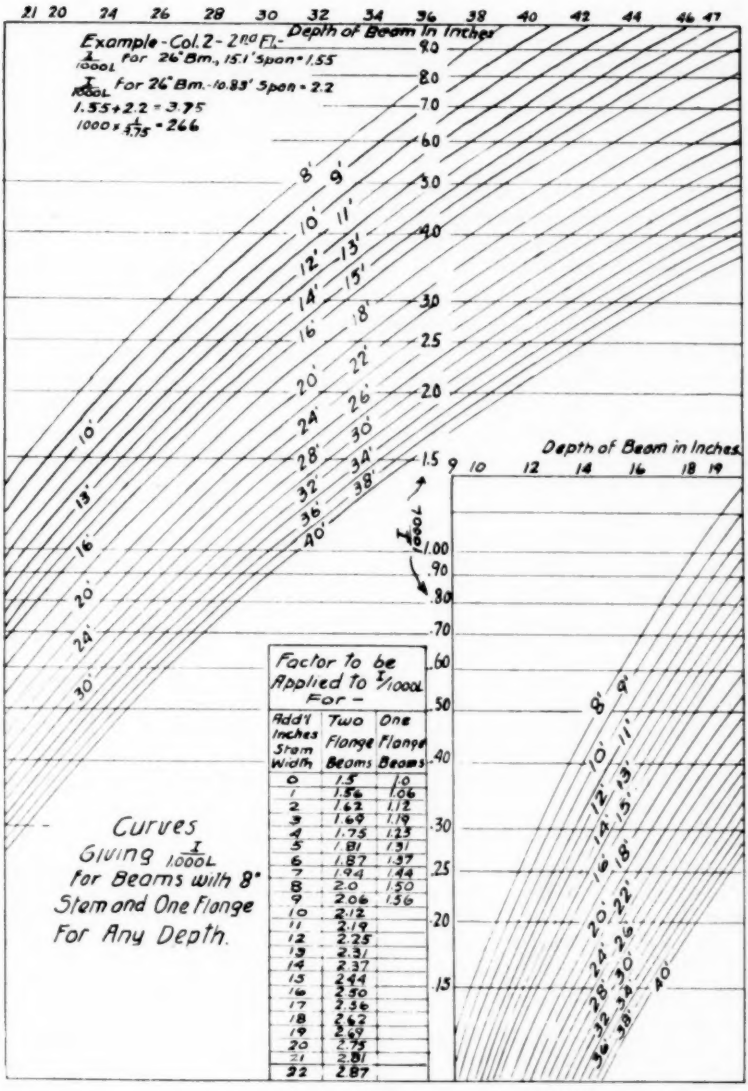

Depth of Beam in Inches
Example - Col. 2 - 2nd Fl. -
1/1000L for 26" Bm., 15.1' Span = 1.55
I/1000L for 26" Bm. - 10.83' Span = 2.2
1.55 + 2.2 = 3.75
1000 × 1/3.75 = 266
Depth of Beam in Inches
Curves Giving I/1000L for Beams with 8" Stem and One Flange For Any Depth.
Factor to be Applied to I/1000L For —
Add'l Inches Stem Width | Two Flange Beams | One Flange Beams
0 1.5 1.0
1 1.56 1.06
2 1.62 1.12
3 1.69 1.19
4 1.75 1.25
5 1.81 1.31
6 1.87 1.37
7 1.94 1.44
8 2.0 1.50
9 2.06 1.56
10 2.12
11 2.19
12 2.25
13 2.31
14 2.37
15 2.44
16 2.50
17 2.56
18 2.62
19 2.69
20 2.75
21 2.81
22 2.87

FIG. 3

probably quite small. The most serious question is whether the difference between $\frac{16d^3}{12}$ and $\frac{24d^3}{12}$ truly represents the difference between the stiffness of spandrel and interior beams. After much computation the I's here used seem to the writer to give the best relative results for varying depths, widths, and flanges.

Tables 2 and 3 show the calculation work tabulated for Eddystone Homes, a 22-story reinforced concrete building in Chicago. See Fig. 4. The wind action on this building is so intricate that we regard it as a fair sample of the worst case. It has been worked out here for the second, the tenth and the seventeenth floors. The columns change size at the ninth and at the sixteenth floors, and since most of the columns are rectangular and the beam span change is unusually large, the stiffness factors and the moments are not proportionate to those at the second floor. Probably in most buildings with square columns and uniform depth of spandrels, two, instead of three calculations would be sufficient, and the second one might be made only partially. The complete tabulation and calculation of these three floors give us a very interesting study of the distribution at any one floor, and the change of distribution caused by the framing changes in higher stories.

In Tables 2 and 3 and the headings explain themselves as terms in the formula of Fig. 1. The multipliers 1000 and 1,000,000 are used for convenience in tabulating.

When the column $\frac{1}{R}$ is filled out we can solve the equation $K = M/\frac{1}{R}$ to get the average value of K. In a symmetrical building the average K would be the true K; that is the deflection of the floor would be the same at all points along the line of exposure. If, however, the building is stiffer at one end than at the other, there would be less deflection at that end, and K, which is D times a constant, will vary from end to end. We may regard the stiffness units as similar to fibres in a beam and compute the center of gravity of the stiffness, (See Fig. 5). From the eccentricity thus determined we compute the moment of eccentricity, computing now the moment of inertia of the stiffness units. We can compute the moment stress on the fibres at any point, or on the stiffness units at any point, located and identified as in Fig. 5. If we are regarding the stiffness units as fibres, the average K is to be regarded as the direct fibre stress, and the moment fibre stresses are to be added to or subtracted from the average K to give the true K at braced points along the exposure.

It seems reasonable to regard the axis of the beams as the location of the moment resistance, assembling the groups of joints acting along one line, where a number of beams are nearly in line, an approximation by inspection to the average line of action is probably quite close enough. Where a number of beams of small stiffness are over several feet apart, there is no useful purpose served in listing each as a separate force. The error in an approximate location of their center of gravity causes very small error in the important moments, and the moments in the shallow beams themselves are not important.

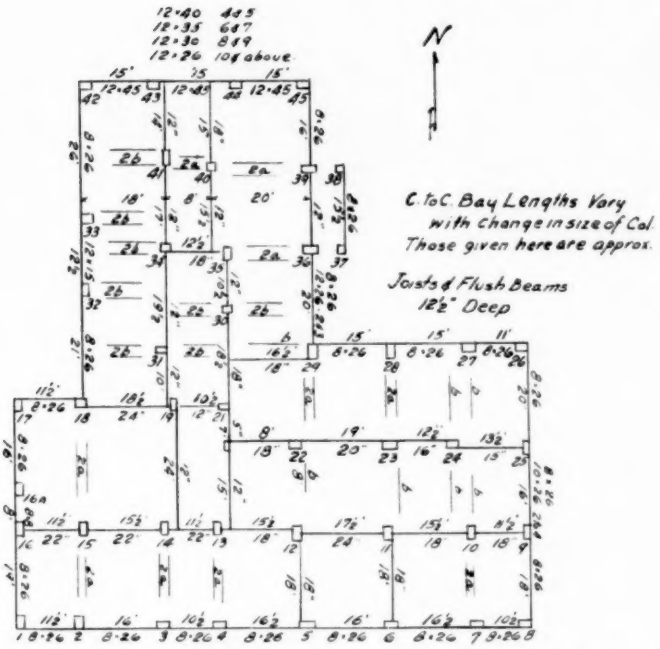


FIG. 4—EDDYSTONE BELMONT BLDG., SHERIDAN RD. AND BELMONT AVE. MCNALLY AND QUINN, ARCHITECTS. SMITH AND BROWN, ENGINEERS, INC. DIAGRAM OF FRAMING FOR DETERMINING STIFFNESS FACTORS FOR WIND STRESSES

The true K's for each group of joints having been determined we return to Tables 2 and 3 and multiply these values into the quantities $\frac{1}{R}$ to get the total moments at each of the joints.

These total moments are divided between the column sections above and below the floor, in proportion to the shears, except where there is great difference in the stiffness of the column above and below the floor. Where this disparity exists, the point of contra-flexure above is below the middle and the point of contraflexure below is below the middle; and the column below should be assigned moment in proportion to its stiffness. The column below should be assigned a moment somewhat greater than the total moment multiplied by the ratio of the shears. The ratio of the shears is found from the general stress diagram. For the second floor East and West, the shears above and below are 470,000 and 492,000. The proportion of moment above would be $\frac{470}{962} = .49$.

FIG. 5—CALCULATION OF ECCENTRICITY AND OF CORRECTIONS FOR K

TABLE 2—EDDYSTONE BELMONT BLDG., SECOND FLOOR—WIND EAST AND WEST

Col. Size	Col. Load	Mill.h / 2Ic	Col. No	L	L'	Ib / 1000L	Ib' / 1000L'	Million Ib/L + Ib'/L'	Million xR	l/R	True K	M in Joint	%	m Left	m' R't	Max m Left	Rot. for R't
17x30		308	1	10.8'	–	2.2	–	455	763	1305	724	95	–	95	–	91	–
"		273	2	15.1'	10.8'	1.55	2.20	266	539	1860	"	135	58	56	79	61	83
"		77	3	9.2'	15.1'	2.54	1.55	245	322	3100	"	225	62	140	85	140	77
"		77	4	15.2'	9.2'	1.54	2.54	246	323	3090	"	224	62	84	140	87	140
17x34		51	5	14.6	15.2	1.61	1.54	317	368	2715	"	197	51	100	97	100	93
"		51	6	15.5	14.6	1.51	1.61	320	371	2695	"	195	52	95	100	89	100
17x30		77	7	9.2	15.5	2.53	1.51	247	324	3090	"	224	63	150	74	150	79
"		92	8	–	9.2	–	2.35	392	484	2060	"	150	–	–	150	–	150
18x32		240	9	–	10.7	–	.52	1920	2160	462	699	32	–	–	32	–	32
17x34		184	10	10.7	14.8	.52	.37	1120	1304	764	"	54	38	31	23	31	23
17x42		138	11	14.8	16.7	.37	.21	1710	1848	540	"	38	64	24	14	24	14
"		138	12	16.7	14.8	.21	.40	1640	1778	561	"	39	64	14	25	14	25
17x34		176	13	14.8	10.3	.40	.52	1095	1271	787	"	55	57	24	31	24	31
17x36		174	14	10.5	14.8	.52	.37	1120	1294	771	"	54	58	31	23	31	23
17x42		138	15	14.8	10.8	.37	.32	1120	1258	797	"	56	58	24	3	24	32
18x32		240	16	10.8	–	.52	–	1920	2160	463	"	32	–	32	–	32	–
17x30		300	17	11.5	–	2.06	–	486	786	1270	66.7	84	–	84	–	90	–
32x32		94	18	17.7	11.5	.37	2.06	411	505	1975	"	132	84	21	111	21	103
17x36		173	19	9.6	17.7	.59	.37	1030	1203	830	"	55	61	34	22	35	22
17x30		305	20	6.9	9.0	.79	.38	860	1165	858	67.8	58	63	39	19	41	19
"		90	21	–	9.6	–	.59	1680	1770	565	66.7	38	–	–	38	–	37
20x30		55	22	17.5	6.9	.37	.79	860	915	1093	67.8	74	68	24	50	24	47
"		63	23	14.7	17.5	.39	.37	1310	1373	730	"	49	51	25	24	25	24
17x34		54	24	12.7	14.7	.44	.39	1200	1254	798	"	54	53	29	25	29	25
17x36		196	25	–	12.7	–	.44	2270	2466	405	"	27	–	–	27	–	27
17x30		93	26	–	9.2	–	2.55	393	486	2055	65.4	135	–	–	135	–	132
"		80	27	9.2	14.1	2.53	1.66	237	317	3140	"	205	60	124	81	126	76
17x32		207	28	14.1	13.9	1.66	1.70	297	504	1980	"	130	51	64	66	70	72
24x30		88	29	13.9	23.0	1.70	.24	515	603	1660	"	109	87	96	13	89	13
17x34		59	30	23.0	13.0	.18	.28	2150	2209	451	64.2	29	61	11	18	11	18
17x30		82	31	15.0	23.0	.28	.24	1920	2082	480	65.4	31	54	17	14	17	14
17x36		173	32	11.0	–	.31	–	3200	3373	296	64.2	19	–	19	–	19	–
21x32		84	33	24.0	–	.18	–	5600	5684	176	628	11	–	11	–	11	–
26x26		84	34	12.1	23.0	.47	.24	1410	1494	670	"	42	66	28	14	27	14
17x30		302	35	15.8	12.1	.27	.47	1350	1652	606	"	38	63	14	24	15	25
"		88	36	3.3	15.8	.04	.27	3260	3348	299	"	19	88	2	17	2	17
17x17		317	37	–	3.3	–	.04	26,300	26,800	37	"	2	–	–	2	–	2
"		317	38	–	3.3	–	.04	26,300	26,800	37	61.0	2	–	–	2	–	2
17x30		88	39	3.3	18.6	.04	.23	3770	3,858	259	"	16	88	2	14	2	14
23x23		140	40	18.6	7.6	.23	.57	1260	1,400	715	"	44	71	13	31	13	31
17x30		292	41	7.6	26.0	.37	.16	1,365	1,657	605	"	37	78	29	8	29	8
17x34		50	42	12.8	–	10.80	–	92	142	7015	589	420	–	420	–	373	–
18x32		67	43	13.7	12.8	10.0	10.8	48	115	8700	"	511	52	246	265	246	295
"		67	44	12.8	13.7	10.8	10.0	48	115	8700	"	511	52	265	246	280	246
17x30		85	45	–	12.8	–	10.8	92	177	5630	"	332	–	–	332	–	314
									ΣK = 77095			ΣM = 5010					

Average K = $\frac{\Sigma M}{\Sigma K}$ = 65.0
Mom. at Ends = ½ ms. corrected + or – by fraction –(Diff. in Moms)/2 x Sum
Correction at (1) 95-79 = 16 95+79 = 174 16/348 = .046 95 x .046 = 4 (minus)

TABLE 3—EDDYSTONE BELMONT BLDG., SECOND FLOOR—WIND NORTH AND SOUTH

Col. Size	Col. Load	M/k/2Ic	Col. No	L	L'	Ib/1000L	Ib'/1000L'	Million Ip+Ip'	Million xR	1/R	true K	M in Joint	%	m Left	m' R't	Rev. m Left	Rev. m' Rt
17×30		90	1	—	17.6'	—	1.35	740	830	1210	1178	144	-	-	144	-	145
"		86	2	—	17.8'	—	.35	2850	2936	340	1190	40	-	-	40	-	40
"		280	3	—	18.5'	—	.34	2950	3230	310	1206	37	-	-	37	-	38
"		280	4	—	18.4'	—	.34	2950	3230	310	1217	37	-	-	37	-	38
17×34		230	5	—	16.8'	—	.33	3020	3250	307	1234	38	-	-	38	-	38
"		230	6	—	16.8'	—	.33	3020	3250	307	1247	38	-	-	38	-	38
17×30		280	7	—	17.2'	—	.37	2700	2980	334	1263	42	-	-	42	-	43
"		320	8	—	17.1'	—	1.32	760	1080	925	1274	118	-	-	118	-	123
18×32		70	9	17.1'	14.5'	1.32	1.62	340	410	2440	1274	311	55	140	171	134	173
17×34		60	10	17.2'	15.6'	.37	.40	1300	1360	735	1263	93	52	45	48	44	47
17×42		32	11	16.8	24.0	.33	.36	1450	1482	674	1247	84	52	40	44	40	44
"		32	12	16.8	—	.33	—	3020	3052	327	1234	40	-	40	-	40	-
17×34		58	13	18.4	20.0	.34	.23	1750	1808	553	1217	67	60	40	27	40	27
17×36		52	14	18.5	24.1	.34	.27	1630	1682	593	1206	72	56	40	32	40	32
17×42		32	15	17.8	20.9	.35	.30	1530	1562	639	1190	76	54	41	35	41	35
18×32		70	16	17.6	6.5	1.35	.12	680	750	1333	1178	158	92	146	12	145	12
17×26		139	16A	6.5	14.1	.12	1.66	560	699	1425	1178	169	93	12	157	12	160
17×30		88	17	14.1	—	1.66	—	600	688	1450	1178	171	-	171	-	167	-
32×32		33	18	20.4	21.4	.30	1.07	730	763	1310	1190	156	78	35	121	35	118
17×36		52	19	24.1	9.4	.27	.48	1330	1382	722	1206	98	64	32	36	32	34
17×30		88	20	20.0	6.3	.23	.48	1400	1488	673	1217	82	67	27	55	27	33
"		300	21	6.3	17.9	.48	.32	1250	1550	645	1217	79	60	48	31	30	31
20×30		155	22	24.0	24.0	.22	.27	2030	2185	458	1234	56	55	25	31	25	31
"		132	23	24.0	25.0	.36	.25	1630	1782	560	1247	70	59	41	29	41	29
17×34		230	24	15.6	18.8	.40	.33	1370	1600	625	1263	79	55	43	36	44	36
17×35		55	25	14.5	19.2	1.62	1.22	352	407	2460	1274	315	57	180	135	178	129
17×30		320	26	19.2	—	1.22	—	820	1140	880	1274	112	-	112	-	117	-
"		280	27	18.8	—	.33	—	3020	3300	303	1263	38	-	38	-	38	-
17×32		73	28	25.0	—	.25	—	4000	4073	246	1247	31	-	31	-	31	-
24×30		56	29	24.0	18.9	.27	1.23	667	723	1380	1234	170	82	30	140	30	131
17×36		243	30	17.9	9.9	.32	.47	1270	1515	660	1217	80	59	33	47	33	48
17×30		270	31	9.4	18.4	.48	.24	1380	1650	606	1206	73	67	49	24	51	25
17×36		49	32	21.9	11.0	1.07	2.12	314	363	2760	1190	329	66	111	218	113	207
21×32		122	33	11.0	25.3	2.12	.92	330	452	2220	1190	254	69	177	77	186	80
26×26		84	34	18.4	15.7	.24	.29	1880	1964	510	1206	63	55	29	34	28	34
17×30		88	35	9.9	18.0	.47	.31	1280	1368	730	1217	89	60	53	36	51	36
"		290	36	18.9	14.8	1.23	.18	706	996	1004	1234	123	87	109	16	116	16
17×17		466	37	—	14.8	—	1.60	625	1091	920	1239	114	-	-	114	-	114
"		466	38	14.8	—	1.60	—	625	1091	920	1239	114	-	114	-	114	-
17×30		290	39	14.8	15.3	.18	1.53	585	875	1140	1234	140	89	15	125	15	125
23×23		135	40	18.0	20.0	.31	.28	1700	1835	545	1217	66	53	35	31	33	31
17×30		86	41	15.7	16.0	.29	.28	1750	1836	545	1206	66	-	33	33	33	33
17×34		210	42	25.5	—	.92	—	1080	1290	776	1190	92	-	92	-	88	-
18×32		214	43	16.0	—	.28	—	3570	3784	265	1206	32	-	32	-	32	-
"		210	44	20.0	—	.28	—	3570	3780	265	1217	32	-	32	-	32	-
17×30		310	45	15.3	—	1.53	—	650	960	1040	1234	128	-	128	-	128	-
									$\Sigma 1/R$ =	39378	ΣM =	4815					

Av. $K = \frac{\Sigma M}{\Sigma 1/R} = 122.2$

Revised m & m' obtained as for E & W stresses - See Table 2

The total joint moment is divided between the beams on either side of the column in the proportion of their values of $\frac{I}{1000L}$. In the column marked "%" is given the percentage found by dividing the larger $\frac{I}{1000L}$ by the sum. The correction of these moments is described later.

The moments here computed are those existing at the intersection of the center line of column with the center line of beam. These moments must be reduced to the face of the column and the face of the beam. Beam moments must be multiplied by the ratio one-half the span divided into one-half the span less one-half the column width. Similarly, the column moments must be multiplied by the ratio one-half the story height divided into one-half the story height less one-half the beam depth.

ACCURACY OF THIS METHOD

The results show that the assumption that points of contraflexure are at the middle of beams and columns is rarely exactly correct, and is often largely in error. After much recomputation of the joints adjacent to a beam whose end moments are unequal, we have found that if each moment is multiplied by a factor found by dividing the difference by twice the sum of the moments, the positive and negative correction found will make the corrected moments nearly correct.

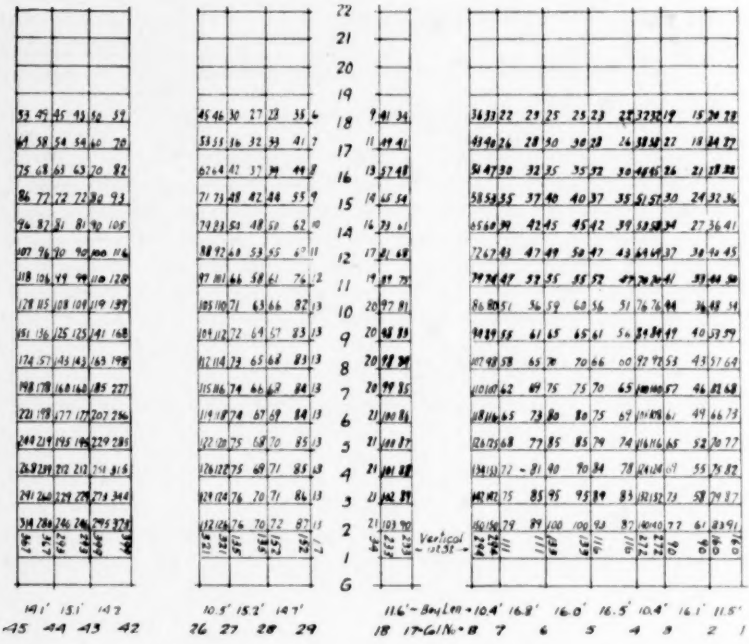
Where the column size changes, points of contraflexure above and below are not at mid-height, and in the rare case where the column wind stress at such a floor is critical, allowance must be made for additional moment at the upper end of the column below and at the upper end of the column above.

The wind moments for the various floors are computed for the full story height. In distribution of the moments the stiffness factors are based on beam lengths equal to the center to center of columns less one quarter the width of the two columns, and the column height is taken as floor to floor less one quarter the depth of two beams. In very short spans, whatever error this practice involves is magnified in effect. It is our practice, however, to make very short beams shallow.

FINAL SOLUTION FOR VERY UNSYMMETRICAL BUILDINGS

The stresses for Eddystone Belmont, shown on Tables 4 and 5, give a fairly accurate distribution of the moments and vertical

TABLE 4—EDDYSTONE BELMONT BLDG.—WIND EAST AND WEST

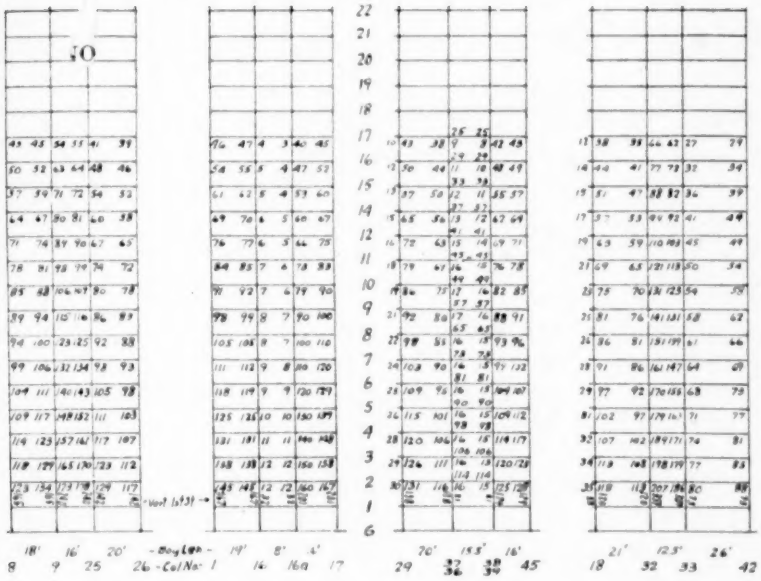


stresses in respect to bending of the beams and columns. Another element of deflection, that due to shortening of columns, must be taken into account.

It is clear that unless the exterior and first intermediate columns have a fibre stress from wind of the same sign and proportionate to their distance from the neutral axis of the bent, so that the floors remain in the same plane, secondary stresses are set up in the joints. These secondary stresses will remove vertical load from the exterior columns, and put loads of the same sign and of larger amount, in the intermediate columns. The major part of this transfer takes place in the upper third of the building, and in some cases the beams in the second bay of the bent require reinforcement for a few stories from the top.

In Eddystone Belmont, bay 17-18 is shown having moments of 103,000 ft. lb. and 90,000 ft. lb., and vertical loads in the columns of 199,000 lb. and 233,000 lb. It is quite certain that

TABLE 5—EDDYSTONE BELMONT BLDG.—WIND NORTH AND SOUTH



this bent cannot have the deflection which would result from the column deformation for these stresses. The stresses there shown are much too large. It is possible to make free hand adjustments of these stresses. To describe the adjustments which could properly be made in the stresses of this building is beyond the scope of this paper. I hope to compute a number of crooked buildings, and describe such stress adjustments, so that the designer will have available an example which will show the order of the stress aberrations, and can make an estimate instead of a guess, in the crooked problem before him.

ACCURACY OF THE CONVENTIONAL METHODS

This calculation has been prepared to show that the conventional methods not only leave a chance for error in determining what load comes to each bent, but involve in odd framing, errors in a single bent that are much too large. The labor of these calculations is considerable, but the engineer can readily deter-

mine what a conventional method would give him for moments in any one of these bents, and then determine for himself whether the error is tolerable.

Readers are referred to the JOURNAL *for June, 1931 for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

Economics of Light Weight Concrete in Buildings

*Report of Committee 406, Use of Light Weight Concrete in Buildings**

BY FRANK A. RANDALL, AUTHOR-CHAIRMAN†

In the last few years several aggregates have been developed to produce lighter weight concretes, in addition to cinders which have been used extensively in some localities for many years.

The desirability of an aggregate to produce a lighter weight concrete is usually prompted by relative costs. With unit costs and strengths of the concretes equal, in general of course, the lighter concrete will produce the cheaper frame.

The present study is an attempt to show the comparative costs of the structural frames for concretes of various weights and costs. To cover all buildings in this comparison would be a large undertaking and it was necessary to introduce limitations which are stated in the following paragraphs. The author is of the opinion, however, that the data developed will apply without serious modification to the general problem.

The graphs and tables show how much more can be expended profitably to reduce the weight of concrete.

In using the data submitted herewith it should be borne in mind that an interior panel only is considered. For exterior panels, the effect of reduction in weight of the concrete floor construction is less than for interior panels, for two reasons. First, the spandrel girders and exterior columns carry the wall construction in addition to, usually, only one-half panel of floor construction and, hence, the dead weight of the floor is a smaller portion of the total load carried than for an interior panel.

*Prepared by Mr. Randall, this report was reviewed and in general approved before publication by the critic members of the committee to whom Mr. Randall makes acknowledgment: Frank Brown, of Smith & Brown, Chicago; Emil Praeger, New York City; W. S. Wolfe, of Smith, Hinchman & Grylls, Detroit.

†Structural Engineer, 205 West Wacker Drive, Chicago.

Second, particularly in high buildings, the sizes of the spandrel girders, and, sometimes, of the columns must be increased due to the wind load stresses; and this increase will be greatest for the lightest weight concrete, other conditions being the same.

A similar effect to that last mentioned would take place were the building subjected either to a live load or a wind load in excess of that for which it is designed, that is, the percentage of overstress would be greatest for the lightest weight concrete. The obvious answer is to design for the proper loadings, if possible. This is not a difficult matter as to live loads for the type of buildings considered, namely, residential or office. In the matter of wind loads, the judgment of the designer is required. Building ordinance requirements are minimum requirements, and it might be quite proper to use somewhat higher wind loads for the lighter weights of concrete than for the heavier weights of concrete in order to secure equivalent stiffness of the building.

One reason for the choice of panel size is that this size is used in Part Two, Cost Data in the *Handbook of Reinforced Concrete Building Design* of the American Concrete Institute.[1]

The lower limit of weights of concrete considered in this report is perhaps theoretical. Concrete weighing 60 lb. per cu. ft. has been used in 4-in. slabs spanning 5 ft. with a design live load of 100 lb. per sq. ft. and concrete weighing between 75 and 100 lb. per cu. ft. has been used in long span joist construction.

GENERAL ASSUMPTIONS

Type of Building. The buildings considered are of the residential or office type.

Loading. The loadings in pounds per sq. ft. of floor, used in the calculations, are as follows:

Live load	40
Partitions	20 (with no reduction of the live load on the beams and girders which is equivalent to an increase of the partition load on the beams and girders.)
Floor finish	5
Plaster, direct	5
Plaster on metal lath	10
Composition roofing	5

Panel Size. An interior panel 18 ft. square is considered.

[1] A. C. I., *Proceedings*, Vol. 24, p. 537. 1928.

Story Height. The story height is taken as 11 ft. floor to floor, for all stories including the basement.

Building Code. The unit stresses and regulations of the new proposed Chicago Building Code for fireproof construction are followed. This code embodies, substantially, the 1928 Joint Code and the American Institute of Steel Construction Code. Steel floor beams are designed as composite beams in accordance with the provisions of the new Chicago Code.

Concrete Stresses. The following unit compressive stresses in concrete are used as a maximum:

Floors	2000 lb. per sq. in.
Columns	3000 lb. per sq. in.

Unit Costs. The following unit costs of materials in place are assumed:

Reinforcement bars	$0.035	per lb.
Reinforcement mesh	0.045	per lb.
Spirals	0.05	per lb.
Structural steel	0.04	per lb.
Formwork	0.25	per sq. ft.
Concrete (substructure)	0.40	per cu. ft.
Excavation and backfill	2.70	per cu. yd.
Wood piles	25.00	each
Caissons (complete)	1.25	per cu. ft.
Plaster on joist construction, excess cost over plaster on solid slabs	0.01	per cu. ft. of building

VARIABLES

Weights of Concrete. Five weights of superstructure concrete are considered ranging from 50 lb. per cu. ft. to 150 lb. per cu. ft. as follows:

Type	Weight
A.	50 lb. per cu. ft.
B.	75 lb. per cu. ft.
C.	100 lb. per cu. ft.
D.	125 lb. per cu. ft.
E.	150 lb. per cu. ft.

Types of Floor Construction. Two types of floor construction are considered, viz.:

F.	Solid slabs 4 in. thick
G.	Steel form 20-in. joists

Types of Frame. Two types of frame are used, viz.:

H.	Concrete
J.	Steel

Building Heights. Six heights of building are considered ranging from five to thirty stories, and basement, viz.:

K.	5 stories and basement
L.	10 stories and basement
M.	15 stories and basement
N.	20 stories and basement
O.	25 stories and basement
P.	30 stories and basement

Foundations. Spread footings at 3000 lb. per sq. ft. are used under the 5 and 10-story buildings; wood piles at 25 tons each, under the 15 and 20-story buildings; and caissons on hard pan (50 ft. high) at 6 tons per sq. ft., under the 25- and 30-story buildings.

Cost of Superstructure Concrete. Three unit costs of superstructure concrete are considered, viz., thirty, forty and fifty cents per cu. ft. in place.

DETAIL ASSUMPTIONS

Stresses. The value of j was taken as equal to 0.875 for slabs, beams and girders. For slabs, the positive and negative moment coefficient was taken as 1/12; for beams, the positive coefficient was taken as 1/16 and the negative coefficient as 1/12. Unit shear was taken at 60 lbs. per sq. in. with the longitudinal reinforcement anchored. The allowable unit pressure under steel column bases was taken as 600 lbs. per sq. in.

Column Loads. The full panel area of 324 sq. ft. was used in computing the column loads, the load on the area occupied by the column being compensated by a reduction of one foot in the length of the column when computing the weight of the column, i. e., by using 10 ft. instead of 11 ft. for the length of the column.

Slabs. For the 4-in. solid slabs, the 18-ft. panel was divided into three equal spans. The slabs were designed for full continuity, using L as the distance center to center of the intermediate beams.

Beams and Girders. All beams and girders were made as shallow as possible, economically. Steel beams were designed for an L center to center of connections. The following allowances in center to center dimensions of steel framing were made: for beams, 6 in.; girders, 15 in. Concrete beams and girders were designed for the clear distance between supports.

Concrete Columns. Spirals were used in all cases. The sizes were kept constant in each group of five stories. For the type A and B concretes throughout all stories and for the type C, D and E concretes in the upper five stories, the column design was based on 2,000-lb. concrete; for the balance of the columns, 3,000-lb. concrete was used. Three spacers weighing ¾ lb. per ft. were used for core diameters from 10 in. to 19 in., inclusive, and four spacers for larger columns.

Steel Covering. The top flanges of steel beams were placed 3 in. below the top of the concrete slab, with 2-in. covering on the sides and bottoms. For the concrete slabs a minimum covering of ¾ in. was used on the bottom, and 1 in. on the top of the reinforcement steel. For the concrete joist construction a minimum covering of 1½ in. was used on the bottom and 1 in. on the top. For the upper five stories of the steel columns the minimum covering was taken at 2 in., and for the remainder a minimum covering of 3 in., in which latter case, only, the increased unit stresses for combination columns were used.

Weights of Structural Steel. To make allowance for details and rivets, steel beams were figured center to center in addition to the weight of the connection angles; and one foot was added to the column story height to include column splices, rivets and other details.

Excavation. For spread footings and concrete pile caps, 3 in. was included in the excavation outside of the finished concrete.

DESIGNS

The designs involved 20 roof panels, 20 floor panels, 620 columns and 120 foundations, all different. Consistency of design was kept in mind at all times in order to produce as truly comparable results as possible.

ESTIMATES

After the designs were completed, a careful estimate of all quantities was made to which were applied the unit costs. In the appendices are tabulated the quantities of material for each of the 120 conditions. Appendix 1 is a tabulation of the total quantities of materials in the floors, columns and foundations with the 4-in. solid slab and concrete frame. Appendix 2 is for

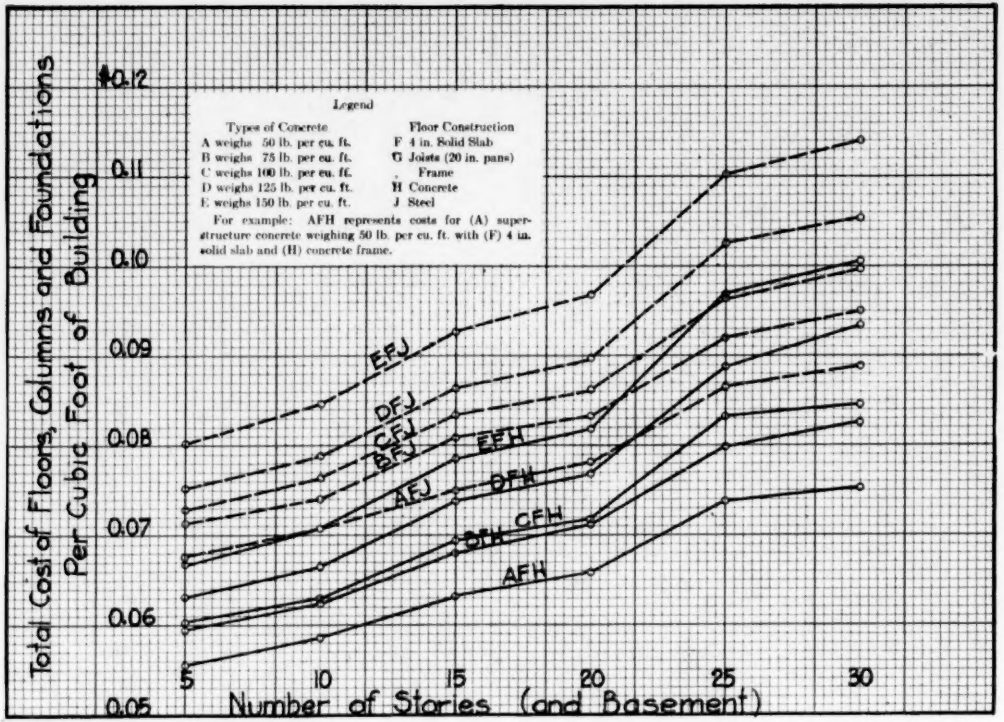


FIG. 1—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT

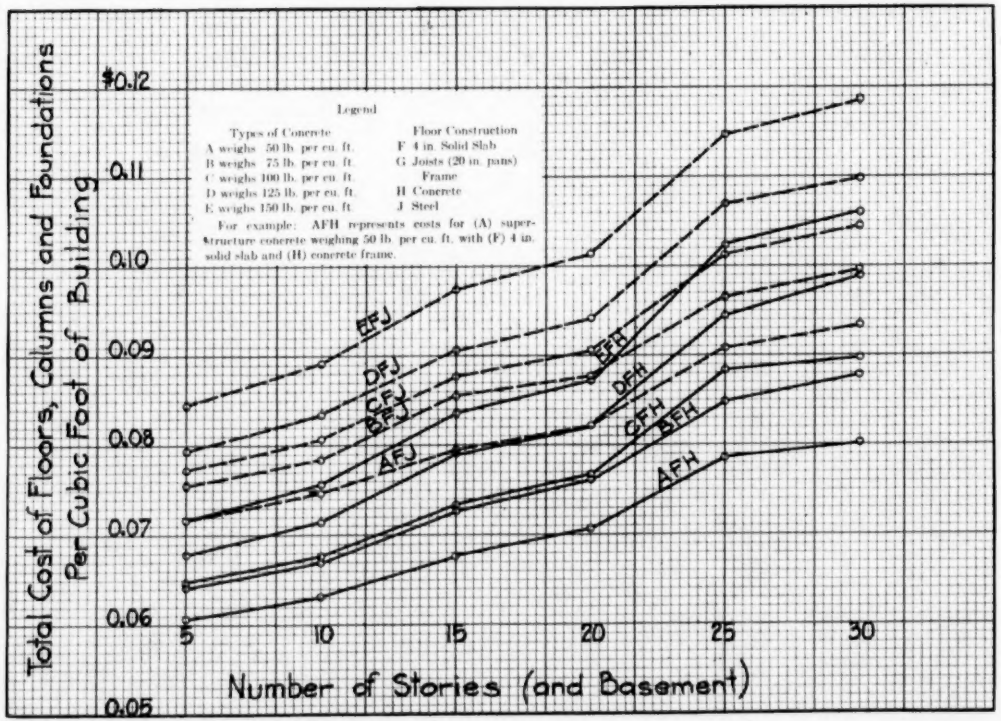


FIG. 2—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT

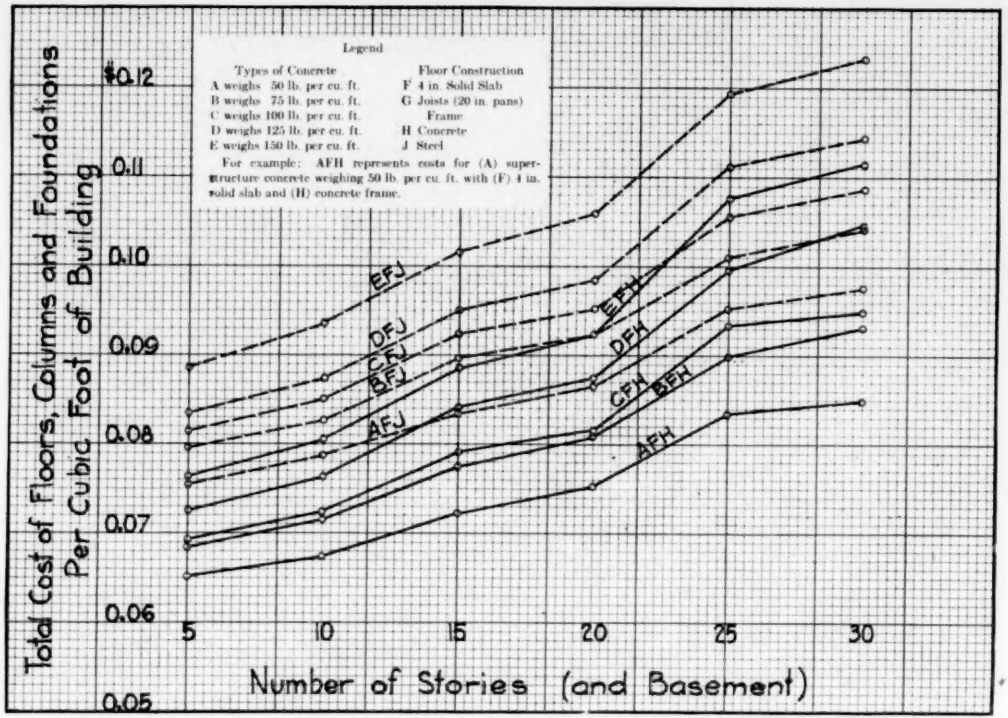


FIG. 3—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT

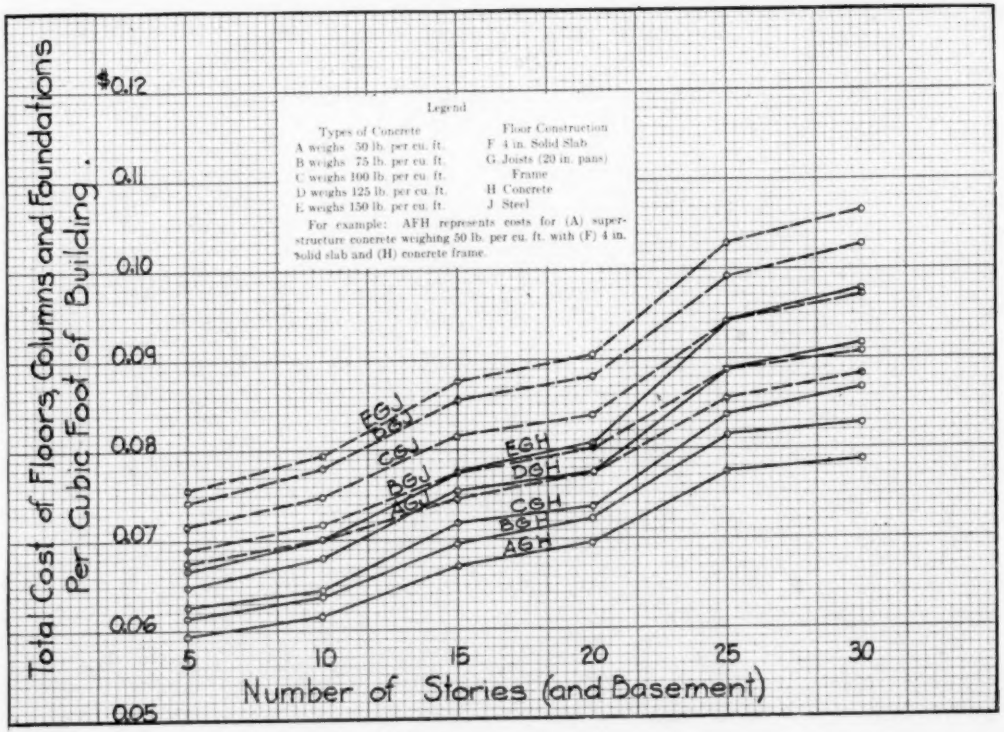


FIG. 4—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT

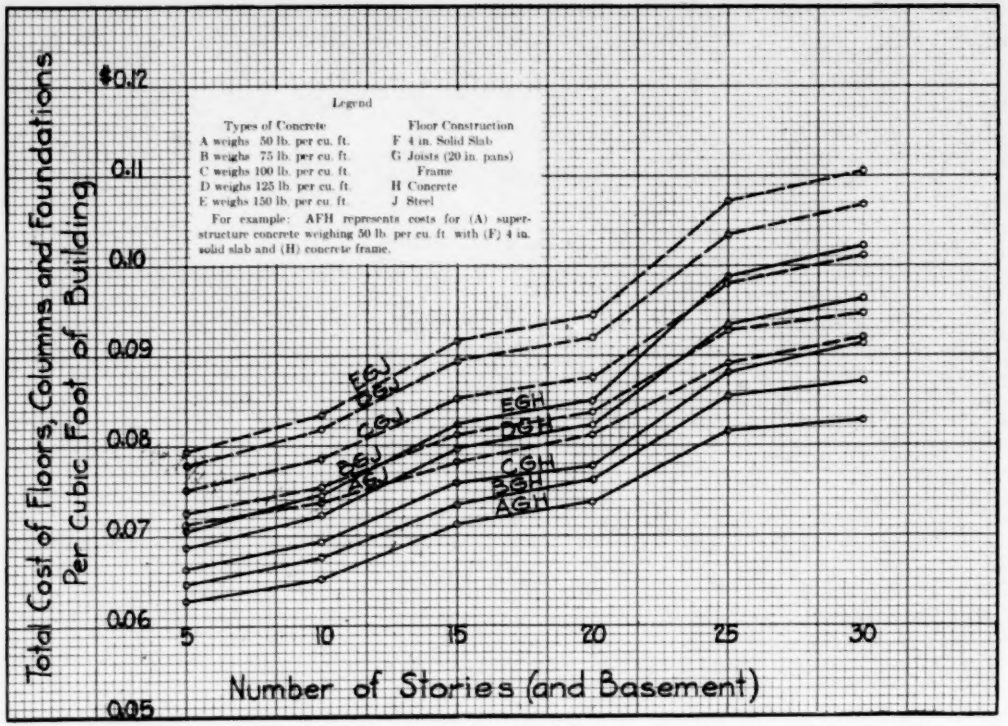


FIG. 5—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT

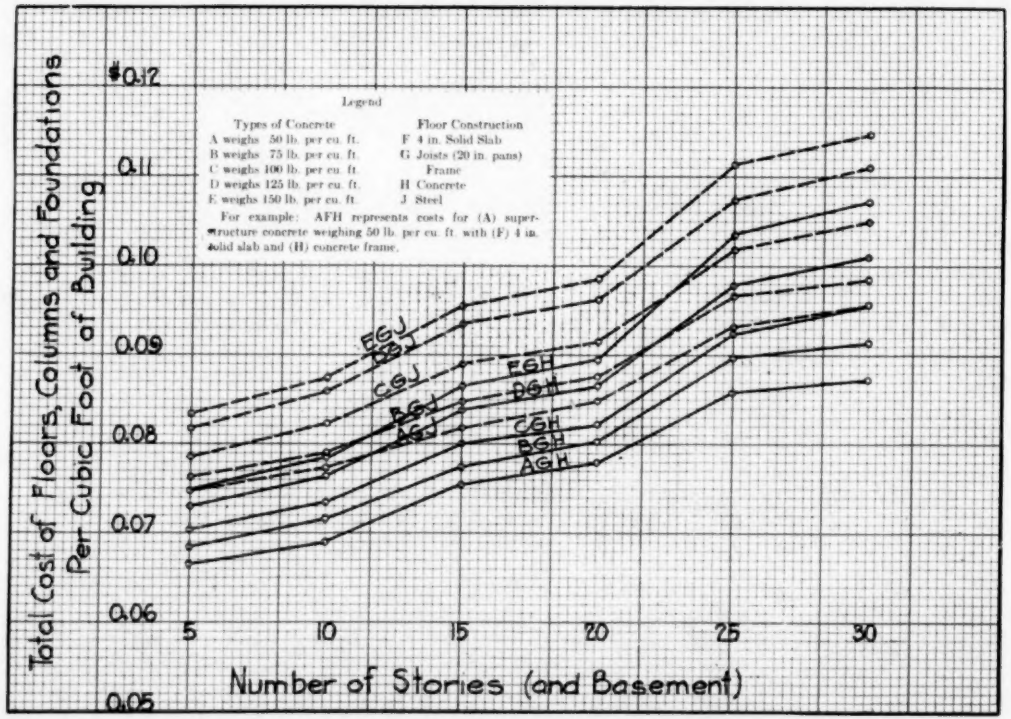


FIG. 6—COMPARATIVE UNIT COSTS OF STRUCTURAL FRAMES — SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT

TABLE 1—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 30¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
			Concrete Frame (H)				
			4-In. Solid Slab (F)				
AFH	50	$0.0562	$0.0586	$0.0631	$0.0659	$0.0738	$0.0752
BFH	75	.0595	.0623	.0680	.0712	.0798	.0827
CFH	100	.0602	.0629	.0694	.0717	.0832	.0845
DFH	125	.0631	.0664	.0739	.0769	.0889	.0934
EFH	150	.0669	.0708	.0785	.0819	.0969	.1004
			Joists (20-in. pans) (G)				
AGH	50	.0593	.0615	.0670	.0696	.0773	.0786
BGH	75	.0612	.0638	.0695	.0721	.0813	.0828
CGH	100	.0626	.0654	.0718	.0737	.0838	.0868
DGH	125	.0649	.0680	.0753	.0777	.0887	.0916
EGH	150	.0667	.0701	.0778	.0804	.0941	.0974
			Steel Frame (J)				
			4-In. Solid Slab (F)				
AFJ	50	$0.0676	$0.0706	$0.0750	$0.0781	$0.0865	$0.0889
BFJ	75	.0713	.0740	.0809	.0832	.0920	.0950
CFJ	100	.0729	.0763	.0833	.0861	.0965	.0997
DFJ	125	.0752	.0789	.0863	.0897	.1025	.1053
EFJ	150	.0801	.0848	.0928	.0969	.1102	.1140
			Joists (20-in. pans) (G)				
AGJ	50	.0675	.0700	.0744	.0773	.0854	.0881
BGJ	75	.0690	.0717	.0773	.0801	.0889	.0908
CGJ	100	.0715	.0749	.0814	.0838	.0941	.0971
DGJ	125	.0742	.0780	.0855	.0880	.0992	.1028
EGJ	150	.0755	.0794	.0876	.0903	.1029	.1063

joist construction with concrete frame; appendix 3, 4-in. solid slab with steel frame; and appendix 4, joist construction with steel frame. These tables are included in order that they may be used with unit costs different from those assumed in this paper.

Superstructure concrete of the various weights was kept separate from foundation concrete and the three different unit prices were used for the superstructure concrete, producing three total costs for each of the 120 buildings. These costs were then reduced to costs per cu. ft. of building, tabulated in Tables 1 to 3, inclusive, and which have been plotted in the following graphs:

Fig. 1 shows the cu. ft. (of building) cost for the 4-in. solid slabs for concrete (H) and steel (J) frame construction based on a unit cost of 30 cents per cu. ft. of the five weights of superstructure concrete.

Fig. 2 is the same as Fig. 1 excepting that the superstructure concrete is priced at 40 cents per cu. ft.

TABLE 2—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 40¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
			Concrete Frame (H)				
			4-In. Solid Slab (F)				
AFH	50	$0.0607	$0.0631	$0.0677	$0.0706	$0.0786	$0.0801
BFH	75	.0641	.0670	.0728	.0761	.0849	.0879
CFH	100	.0648	.0677	.0742	.0767	.0883	.0898
DFH	125	.0679	.0714	.0791	.0822	.0944	.0990
EFH	150	.0717	.0757	.0836	.0872	.1023	.1060
			Joists (20-in. pans) (G)				
AGH	50	.0629	.0652	.0712	.0738	.0816	.0829
BGH	75	.0648	.0676	.0734	.0761	.0855	.0871
CGH	100	.0664	.0694	.0759	.0779	.0881	.0913
DGH	125	.0689	.0722	.0796	.0822	.0933	.0963
EGH	150	.0707	.0744	.0822	.0850	.0988	.1022
			Steel Frame (J)				
			4-In. Solid Slab (F)				
AFJ	50	$0.0717	$0.0747	$0.0793	$0.0824	$0.0908	$0.0933
BFJ	75	.0755	.0784	.0854	.0877	.0965	.0966
CFJ	100	.0772	.0807	.0878	.0906	.1011	.1043
DFJ	125	.0794	.0833	.0907	.0941	.1070	.1099
EFJ	150	.0845	.0892	.0973	.1014	.1148	.1187
			Joists (20-in. pans) (G)				
AGJ	50	.0711	.0737	.0781	.0811	.0892	.0920
BGJ	75	.0726	.0754	.0811	.0838	.0928	.0947
CGJ	100	.0751	.0786	.0852	.0877	.0980	.1011
DGJ	125	.0780	.0819	.0895	.0920	.1033	.1069
EGJ	150	.0794	.0834	.0917	.0944	.1071	.1105

Fig. 3 is a similar set of curves with the concrete priced at 50 cents per cu. ft.

Fig. 4 is for the joist floor construction with the concrete and steel frames and the five weights of concrete costing 30 cents per cu. ft.

Fig. 5 is the same as Fig. 4 excepting the concrete is priced at 40 cents per cu. ft. and

Fig. 6 is the same for 50-cent concrete.

For those who prefer the tabular to the graphic, Tables 1, 2 and 3 show the same data as used in the preceding figures.

Table 1 shows the total cost per cu. ft. of building of the floors, columns and foundations, with superstructure concrete at 30 cents per cu. ft., for the five weights of concrete, the six story heights and for the two types of floor construction with both the concrete and steel frame.

TABLE 3—COST PER CUBIC FOOT OF BUILDING OF FLOORS, COLUMNS AND FOUNDATIONS

(SUPERSTRUCTURE CONCRETE AT 50¢ PER CUBIC FOOT)

Designation	Unit Weight of Superstructure Concrete Lbs. per cu. ft.	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
		Concrete Frame (H)					
		4-In. Solid Slab (F)					
AFH	50	$0.0651	$0.0676	$0.0723	$0.0754	$0.0835	$0.0850
BFH	75	.0687	.0717	.0776	.0810	.0900	.0931
CFH	100	.0694	.0724	.0791	.0816	.0934	.0950
DFH	125	.0727	.0764	.0842	.0875	.0998	.1046
EFH	150	.0765	.0807	.0888	.0925	.1078	.1116
		Joists (20-in. pans) (G)					
AGH	50	.0665	.0690	.0754	.0780	.0859	.0872
BGH	75	.0685	.0715	.0774	.0802	.0897	.0914
CGH	100	.0703	.0734	.0800	.0822	.0924	.0957
DGH	125	.0730	.0764	.0839	.0866	.0979	.1010
EGH	150	.0748	.0786	.0866	.0895	.1035	.1071
		Steel Frame (J)					
		4-In. Solid Slab (F)					
AFJ	50	$0.0758	$0.0789	$0.0835	$0.0867	$0.0952	$0.0977
BFJ	75	.0798	.0828	.0898	.0922	.1011	.1042
CFJ	100	.0815	.0851	.0923	.0952	.1057	.1089
DFJ	125	.0836	.0876	.0951	.0986	.1115	.1144
EFJ	150	.0888	.0937	.1018	.1060	.1194	.1233
		Joists (20-in. pans) (G)					
AGJ	50	.0747	.0773	.0819	.0849	.0931	.0958
BGJ	75	.0762	.0790	.0848	.0876	.0966	.0985
CGJ	100	.0787	.0823	.0890	.0915	.1019	.1050
DGJ	125	.0818	.0859	.0935	.0961	.1074	.1110
EGJ	150	.0833	.0875	.0957	.0986	.1113	.1148

Tables 2 and 3 contain the same data for superstructure concrete at 40 cents and 50 cents per cu. ft., respectively.

DEDUCTIONS

A study of these curves is of interest as disclosing:

1. The effect of increase in building height upon the cost of the structural frame, roughly 1/10 of a cent per cu. ft. of building per story for the range considered.

2. The greater effect of weight of the concrete in the higher buildings, roughly twice as much difference in the 30-story as in the 5-story building.

3. The comparative cost of concrete frame and steel frame, the difference in cost varying from about one cent to 1½ cents per cu. ft. of building.

4. The additional price that profitably may be spent for a lighter weight concrete in order to reduce the cost of the structural

frame. For example, it would be worth while to pay 10 cents more per cu. ft. to secure a reduction from 150-lb. concrete to 130-lb. concrete in a concrete frame building thirty stories high and a reduction to 115-lb. or 100-lb. concrete in a five story building, depending on whether the floor was of solid slab or joist construction. The savings are more favorable in the steel frame building.

We are hopeful that the data developed will prove of value.

(Tabulations constituting appendices 1 to 4 on following pages.)

Readers are referred to the JOURNAL *for June, 1931, for discussion which may develop. Such discussion should reach the Secretary by May 1, 1931.*

APPENDIX 1—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—4-IN. SOLID SLAB WITH CONCRETE FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	3377	7580	12358	18536	25306	33687
	Mesh	lbs.	836	1516	2196	2876	3556	4236
	Spirals	lbs.	421	941	1604	2496	3612	4951
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2767	5119.3	7526.7	10004.7	12289.0	14772.3
	Subst. Concrete	cu. ft.	79	172.9	148	283	..	..
	Excavation	cu. yd.	6.24	12.0	9.0	17.9	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	955	1087
	Super. Concrete	cu. ft.	950	1778	2644	3549.7	4486.7	5453.4
75	Bars	lbs.	3887	8715	14232	21418	29465	39239
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	410	962	1731	2810	4095	5657
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2884	5350.3	7909.0	10497.3	12882.3	15502.3
	Subst. Concrete	cu. ft.	100.4	219.3	227	367	..	..
	Excavation	cu. yd.	7.7	14.7	14.3	23.1	..	..
	Piles	one	..	..	10	14	..	..
	Caissons	cu. ft.	..	..	..	..	1087	1342
	Super. Concrete	cu. ft.	977.1	1835.4	2745.0	3697.6	4687.8	5722.2
100	Bars	lbs.	4509	9796	15643	23193	31675	42201
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	402	950	1720	2798	4108	5715
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2804.3	5195.0	7674.7	10157.3	12465.7	14995.7
	Subst. Concrete	cu. ft.	119.9	251.4	283	367	..	..
	Excavation	cu. yd.	8.8	16.4	17.9	23.1	..	..
	Piles	one	..	..	12	16	..	..
	Caissons	cu. ft.	..	..	..	..	1342	1505
	Super. Concrete	cu. ft.	984.5	1858.2	2772.8	3734.3	4744.7	5783.4
125	Bars	lbs.	4799	10568	16856	25448	34983	47103
	Mesh	lbs.	986	1816	2646	3476	4306	5136
	Spirals	lbs.	461	1068	1991	3210	4746	6649
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2862.3	5312.0	7864.0	10430.0	12755.7	15352.3
	Subst. Concrete	cu. ft.	149.1	323.1	367	490.0	..	..
	Excavation	cu. yd.	10.6	20.2	23.1	30.4	..	..
	Piles	one	..	..	14	19	..	..
	Caissons	cu. ft.	..	..	..	..	1505	1936
	Super. Concrete	cu. ft.	1032.5	1957.5	2938.4	3969.6	5053.0	6170.3
150	Bars	lbs.	5816	12818	20332	30387	41694	55829
	Mesh	lbs.	991	1826	2661	3496	4331	5166
	Spirals	lbs.	469	1121	2160	3506	5206	7283
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2988.3	5548.7	8196.3	10881.0	13304.7	16024.7
	Subst. Concrete	cu. ft.	172.9	378.9	367	537	..	..
	Excavation	cu. yd.	12.0	23.2	23.1	32.6	..	..
	Piles	one	..	..	16	21	..	..
	Caissons	cu. ft.	..	..	..	..	1779	2143
	Super. Concrete	cu. ft.	1032.6	1951.7	2928.7	3957.9	5045.1	6185.8

APPENDIX 2—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—JOISTS WITH CONCRETE FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	4114	8602	14265	21028	28383	37349
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	397	917	1580	2472	3588	4927
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2342.0	4334.3	6381.7	8499.7	10424.0	12547.0
	Subst. Concrete	cu. ft.	79.0	172.6	148.0	283.0	..	..
	Excavation	cu. yd.	6.24	12.0	9.0	17.9	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	955	1087
	Super. Concrete	cu. ft.	781.1	1467.6	2387.6	3151.3	3946.3	4770.9
75	Bars	lbs.	4775	10358	16603	24364	33192	43738
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	382	924	1660	2709	3997	5544
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2367.0	4383.3	6494.0	8617.3	10564.0	12720.7
	Subst. Concrete	cu. ft.	94.6	209.8	227.0	346.0	..	..
	Excavation	cu. yd.	7.3	14.1	14.3	22.1	..	..
	Piles	one	..	..	10	13	..	..
	Caissons	cu. ft.	..	..	..	..	1087	1232
	Super. Concrete	cu. ft.	793.6	1495.1	2248.0	3044.2	3873.4	4734.6
100	Bars	lbs.	5093	10831	17148	25068	34116	44889
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	375	885	1621	2669	3955	5516
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2377.3	4411.0	6529.7	8649.0	10584.0	12754.0
	Subst. Concrete	cu. ft.	113.4	251.4	283.0	367.0	..	..
	Excavation	cu. yd.	8.4	16.4	17.9	23.1	..	..
	Piles	one	..	..	12	15	..	..
	Caissons	cu. ft.	..	..	..	..	1211	1505
	Super. Concrete	cu. ft.	820.1	1556.0	2333.0	3153.1	4010.3	4911.7
125	Bars	lbs.	5439	11721	18506	27230	36940	48887
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	398	973	1792	2952	4377	6130
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2432.0	4508.0	6682.3	8879.7	10845.7	13070.7
	Subst. Concrete	cu. ft.	126.6	284.9	346.0	445.0	..	..
	Excavation	cu. yd.	9.6	18.3	22.1	28.2	..	..
	Piles	one	..	..	13	17	..	..
	Caissons	cu. ft.	..	..	..	..	1365	1652
	Super. Concrete	cu. ft.	864.5	1635.8	2461.8	3335.5	4260.4	5217.9
150	Bars	lbs.	5763	12495	19746	29354	40008	53332
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	412	1040	1994	3243	4820	6776
	Struct. Steel	lbs.	..	..	..	..	..	..
	Formwork	sq. ft.	2466.0	4562.7	6757.7	8968.7	10942.7	13197.7
	Subst. Concrete	cu. ft.	149.1	335.7	367.0	490.0	..	..
	Excavation	cu yd.	10.6	21.0	23.1	30.4	..	..
	Piles	one	..	..	15	19	..	..
	Caissons	cu ft	..	..	..	..	1626	1961
	Super. Concrete	cu. ft.	881.0	1665.8	2507.2	3398.0	4346.2	5340.8

APPENDIX 3—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—4-IN. SOLID SLAB WITH STEEL FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	184	443	221	335	..	..
	Mesh	lbs.	836	1516	2196	2876	3556	4236
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	9701	19498	30745	43694	58538	74814
	Formwork	sq. ft.	2786.5	5173.2	7594.5	10070.8	12344.3	14791.8
	Subst. Concrete	cu. ft.	79.0	181.3	148	283	..	..
	Excavation	cu. yd.	8.9	17.2	12.0	22.4	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	971.6	1190.9
	Super. Concrete	cu. ft.	872.9	1635.0	2417.3	3218.0	4035.7	4853.8
75	Bars	lbs.	218	534	335	394	..	..
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	10074	20381	32762	46957	63338	81517
	Formwork	sq. ft.	2918.0	5350.5	7899.8	10425.8	12752.5	15283.5
	Subst. Concrete	cu. ft.	100.4	219.3	283.0	367.0	..	..
	Excavation	cu. yd.	10.9	20.0	22.4	28.2	..	..
	Piles	one	..	..	11	14	..	..
	Caissons	cu. ft.	..	..	..	..	1087.3	1341.9
	Super. Concrete	cu. ft.	910.2	1712.0	2531.1	3369.9	4223.6	5079.8
100	Bars	lbs.	277	655	335	394	..	..
	Mesh	lbs.	914	1672	2430	3188	3946	4704
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	10720	21900	35363	50955	68907	88979
	Formwork	sq. ft.	2890.5	5364.5	7910.0	10436.8	12771.2	15306.1
	Subst. Concrete	cu. ft.	119.9	251.4	283.0	367.0	..	..
	Excavation	cu. yd.	12.4	22.2	22.4	28.2	..	..
	Piles	one	..	..	12	16	..	..
	Caissons	cu. ft.	..	..	..	..	1232.0	1505.3
	Super. Concrete	cu. ft.	916.5	1722.4	2546.8	3389.1	4249.9	5112.5
125	Bars	lbs.	306	782	384	511	..	..
	Mesh	lbs.	986	1816	2646	3476	4306	5136
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	11487	23461	37776	54796	74202	95987
	Formwork	sq. ft.	2908.8	5395.2	7943.3	10496.5	12801.0	15340.2
	Subst. Concrete	cu. ft.	140.5	296.5	346.0	445.0	..	..
	Excavation	cu. yd.	14.0	25.5	27.0	34.5	..	..
	Piles	one	..	..	13	18	..	..
	Caissons	cu. ft.	..	..	..	..	1505.3	1779.5
	Super. Concrete	cu. ft.	905.1	1699.8	2508.2	3335.6	4182.1	5029.5
150	Bars	lbs.	392	908	394	553	..	..
	Mesh	lbs.	991	1826	2661	3496	4331	5166
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	13496	27790	44736	64612	87240	112614
	Formwork	sq. ft.	2935.5	5438.8	8003.5	10579.2	12872.0	15422.8
	Subst. Concrete	cu. ft.	156.7	349.6	367.0	537.0	..	..
	Excavation	cu. yd.	15.3	29.1	28.2	46.0	..	..
	Piles	one	..	..	15	20	..	..
	Caissons	cu. ft.	..	..	..	..	1625.8	1971.1
	Super. Concrete	cu. ft.	926.7	1737.1	2562.5	3405.7	4270.0	5132.8

APPENDIX 4—TOTAL QUANTITIES OF MATERIALS OF FLOORS, COLUMNS AND FOUNDATIONS—JOISTS WITH STEEL FRAME

Concrete Unit Wt.	Material	Unit	Height of Building 5S + B	10S + B	15S + B	20S + B	25S + B	30S + B
50	Bars	lbs.	1966	3755	5083	6737	7942	9482
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	6588	13401	21820	31668	43449	56799
	Formwork	sq. ft.	2366.5	4393.2	6456.2	8574.2	10487.7	12575.2
	Subst. Concrete	cu. ft.	79.0	181.3	148.0	283.0	..	..
	Excavation	cu. yd.	8.9	17.2	12	22.4	..	..
	Piles	one	..	..	9	12	..	..
	Caissons	cu. ft.	..	..	..	..	954.6	1190.8
	Super. Concrete	cu. ft.	767.0	1439.1	2131.4	2843.4	3570.5	4298.5
75	Bars	lbs.	2140	4124	5601	7346	8642	10322
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	6535	13378	22110	32811	45289	59621
	Formwork	sq. ft.	2344.5	4350.8	6418.2	8479.8	10355.7	12417.5
	Subst. Concrete	cu. ft.	94.6	209.8	227.0	346.0	..	..
	Excavation	cu. yd.	10.3	19.2	18.4	27	..	..
	Piles	one	..	..	10	13	..	..
	Caissons	cu. ft.	..	..	..	..	1068.7	1232.0
	Super. Concrete	cu. ft.	764.1	1435.9	2125.2	2832.2	3555.7	4280.0
100	Bars	lbs.	2404	4643	6232	8176	9667	11552
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	7624	16027	26213	38858	53463	70271
	Formwork	sq. ft.	2384.5	4425.5	6536.0	8627.8	10522.5	12622.2
	Subst. Concrete	cu. ft.	113.4	251.4	283	367	..	..
	Excavation	cu. yd.	11.7	22.2	22.4	28.2	..	..
	Piles	one	..	..	12	15	..	..
	Caissons	cu. ft.	..	..	..	..	1210.9	1480.3
	Super. Concrete	cu. ft.	777.1	1459.2	2160.4	2879.6	3613.9	4353.0
125	Bars	lbs.	2790	5429	7298	9640	11344	13559
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	8228	17248	28478	42102	58407	76855
	Formwork	sq. ft.	2387.8	4438.8	6552.3	8568.8	10534.2	12635.8
	Subst. Concrete	cu. ft.	126.6	284.9	346	445	..	..
	Excavation	cu. yd.	13	24.6	27	34.5	..	..
	Piles	one	..	..	13	17	..	..
	Caissons	cu. ft.	..	..	..	..	1341.9	1652.3
	Super. Concrete	cu. ft.	814.1	1536.9	2274.9	3031.6	3805.4	4582.1
150	Bars	lbs.	2837	5557	7347	9679	11383	13598
	Mesh	lbs.	428	784	1141	1497	1854	2210
	Spirals	lbs.	..	..	..	..	..	..
	Struct. Steel	lbs.	8408	17871	29797	44292	61621	81399
	Formwork	sq. ft.	2411.8	4473.2	6611.3	8724.5	10601.3	12715.5
	Subst. Concrete	cu. ft.	140.5	296.5	367	445	..	..
	Excavation	cu. yd.	14	25.5	28.2	34.5	..	..
	Piles	one	..	..	14	18	..	..
	Caissons	cu. ft.	..	..	..	..	1480.3	1779.5
	Super. Concrete	cu. ft.	832.7	1572.1	2325.2	3097.3	3886.3	4677.6

Discussion of the Paper by Helmer Swenholt:

"Design and Construction of Bonnet Carre Spillway"*

BY C. E. GRUNSKY†

There are some features of the Bonnet Carre Spillway described by Captain Swenholt, which might well be further elucidated for the benefit of the engineering profession. Among these are the following:

Why, for example, in view of the infrequent occurrence of the large discharge, which is anticipated about once in five years, was the spillway not made longer? A maximum overflow depth of 4 or 5 ft. would have been more desirable than 8 ft. Moreover it might even have been possible to secure the desired maximum flow at elevation 24 ft. over a weir crest without controlling the flow with gates or needles and yet without excessive premature functioning of the spillway. Operation costs could have been thereby materially reduced. The results of any studies made of such an arrangement, if topographic features do not preclude it, would be a welcome addition to the paper.

Undoubtedly there must be good reason for supporting the concrete weir structure on piles. Under ordinary conditions this would seem to the writer an undersirable arrangement because the separation of the ground from the base of the pile supported structure by settlement may take place and thereby the piping out of the soil from under the structure is facilitated and maximum uplift is invited. Generally speaking and entirely without personal knowledge of, and, perhaps, without application to conditions at the spillway site, it may be stated that such structures should preferably rest directly with their full weight upon the underlying ground. It is good practice moreover to add weight

†Consulting Engineer, San Francisco.
*Journal A. C. I., November 1930; A. C. I. *Proceedings*, Vol. 27, p. 243.

by providing an overhead sand box if there is considerable difference in elevation between the high water on one side of the spillway and the low water stage on its other side, so that by heavy loading the underlying material will be compacted and the danger of piping out the support will be reduced. Furthermore, it may frequently be advisable to place such a structure on a layer of pervious material, such as sand, so that percolating waters which get past the upper lines of sheeting and of cutoff walls may find easy passage to a down-stream outlet.

The line of steel sheet piling close to the upstream edge of the weir affords it a maximum amount of support along or near its upstream edge. Longitudinal cracking of the floor along the trench feature, shown at the face of the weir, and into the piers may result if there should be more settlement of the piles than of the sheeting. Some explanation would be desirable as to this location of the sheeting in preference to a location under the crest line of the weir where it would have been equally effective in checking the underflow. Again referring to ordinary conditions and to a structure without pile support, the sheeting would then, it seems to the writer, more logically be placed just above the weir structure with concrete resting against it and not upon it.

The reason for the second line of steel piling at the stilling basin is not clear. Something at that point in the nature of protection against under-cutting in case of a failure of the flexible concrete mat and rock fill may be desirable, but the sheeting, as described, functioning to retard underflow, will build up pressure under the fore-apron and this pressure, unless measures for adequate relief have been provided, might float the fore-apron. Under an extreme, and probably unwarranted assumption, that water might find more obstruction to flow at the down-stream line of sheeting than at the upstream line, the uplift due to a difference in head of some 12 feet just before a release of the needles could easily be sufficient to lift and crack the apron.

The engineering profession is under particular obligations to Captain Swenholt for his clear and concise description of this spillway which, because of the great volume of water to be handled, marks a forward step in flood control practice. Later accounts of its effect on flood stages will be eagerly looked for by all hydraulic engineers.

Discussion of Paper by L. J. Mensch:

"Composite Columns"*

BY DAVID MOLITOR†

Some years ago the writer reviewed an article published in 1911, by Dr. F. Emperger on "A new use of cast iron in columns and arch bridges" describing a rather exhaustive series of tests from which he deduces the following conclusions:

1. That the strength of reinforced concrete columns was increased only slightly by vertical bars, while the hooping was mainly responsible for the enhanced carrying capacity.
2. Composite columns, consisting of a steel or cast iron core and encased in a concrete envelope with vertical reinforcement and hooping, could be designed to give exceptional results but depend mostly on the hooping for their greater strength.
3. Filling the interior of the cast iron cylinder with concrete materially diminished the ultimate strength of the encased column. A marked reduction in strength also, followed when octagonal or square spirals were substituted for the round form.

The form of column giving the highest strength according to Emperger, consisted of a hollow cast iron cylinder surrounded by a concrete encasement containing very little vertical reinforcement, but considerable spiral steel, as shown in Fig. 1.

Without attempting any further description of these experiments, it is at once apparent that the author's test columns differ from those of Emperger in that the hollow core feature was disregarded and only solid sections were tested, giving some rather erratic results depending on whether the hooping was capable or incapable of acting.

From all the test data on composite columns examined by the writer, it would appear that the element of weakness in such columns, as in ordinary hooped concrete columns, is due to insufficient hooping material to resist lateral bursting. No

*Journal A. C. I., Nov. 1930, A. C. I. *Proceedings*, Vol. 27, p. 263.
†Structural Engineer, Detroit.

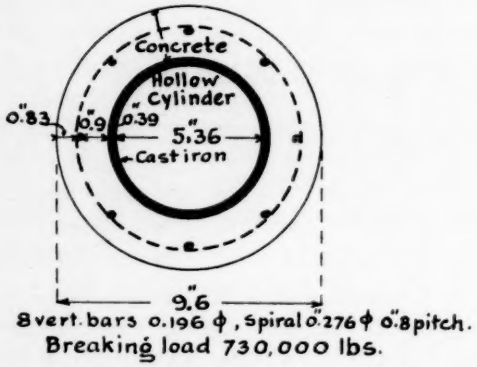


FIG. 1

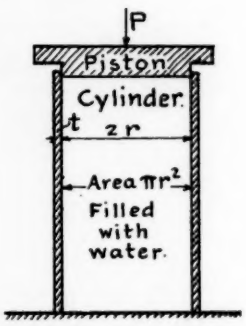


FIG. 2

matter how much core material is supplied, if the hoop strength is not properly provided, the column fails under a low test load.

To illustrate this point, let the cylinder Fig. 2, represent a hollow column as of spirally welded pipe, filled with a liquid, as water, and supported on a base with water-tight connection. On top of the cylinder is a piston acting as a column cap and fitting into the cylinder with water-tight joints but not otherwise transferring any vertical load to the tube. Disregarding details and assuming absolutely tight joints, this is at once a column capable of sustaining any load P by stressing the cylinder only in hoop tension. Let F represent the allowable working tensile stress in pounds per sq. inch, and referring to Fig. 2.

Then the unit pressure on the cylinder wall would be $p = \frac{P}{A} = \frac{P}{\pi r^2}$ and the hoop tension pr would require a thickness of metal

$$t = \frac{pr}{f} = \frac{P}{f\pi r}.$$

The required steel area $= \frac{2\pi r\, P}{f\pi r} = \frac{2P}{f}$ = constant for any given load P.

Hence for $P = 1{,}000{,}000$ lbs. and $f = 18{,}000$ lbs., the steel area would be 112 sq. ins., requiring a cylinder of any diameter with a corresponding thickness t to give this area. Thus for $t = 1$ inch, the diameter would be 35 inches, and the pressure on the water would be 1000 pounds per sq. inch.

In this example we are dependent on a frictionless liquid which transmits the full pressure to the cylinder wall. Now substitute concrete for the liquid and note that most of the lateral pressure is absorbed by internal friction or stress, and only a small fraction of the unit pressure will remain to exert hoop tension, although in the absence of the tension element the concrete core would burst when the load P attains a certain magnitude.

Therefore, there is no reason why such a column could not be designed for any unit load up to or even exceeding the crushing strength of the concrete and require only a relatively thin enclosing steel cylinder, or spiral, subjected only to tensile stress.

According to this reasoning, which accords with the results obtained from tests, it would appear that the capacity of a composite column does not depend on the amount of vertical steel contained therein, but on how well the concrete is confined within the hooping or tensile shell surrounding the concrete core.

The Emperger cast iron columns substantiate this conclusion to a marked degree, for when the hollow c.i. cylinder is filled with concrete, the strength is materially reduced, because the cast iron shell is not suitable to resist hoop tension.

The best solution of this problem (still to be verified by tests) might be found by providing sufficient steel in the form of heavy spiral steel without appreciable vertical steel, designed entirely to resist tension. For this purpose flat bars would be preferable to round hooping bars and the maximum pitch of the spiral should be such as to prevent the flow of concrete through the open space and yet not so small as to prevent good bond between the core and the insulation.

The several test columns described by Mr. Mensch do not reveal any definite consistency in behavior and we must conclude that the underlying law for the strength of composite columns is still somewhat doubtful.

For columns consisting essentially of a steel core with concrete encasement, the increased strength which may exist is undoubted-

ly due to increased stiffness of the steel rather than to direct carrying capacity of the concrete.

For columns consisting of a concrete core surrounded by steel hooping, the increased strength is a direct function of the amount of hooping provided, because it is the hooping which directly opposes the lateral bursting pressure and prevents failure by diagonal tension.

Readers are referred to the JOURNAL *for June, 1931, for further discussion, including author's closure.*

ABSTRACTS

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of the current literature of cement and concrete

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MATERIALS

AGGREGATES

Two new gravel producing plants in Milwaukee district. GORDON F. DAGGETT. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 52.—Success with two sand and gravel plants in district well supplied with plants before they were built is ascribed to making specially prepared aggregates. At plant of Kohler Bros., Prospect Hill, Wis., sand was not washed originally as it was thought to be clean enough. Later both sand and gravel had to be washed to meet specifications, the sand by means of a double screw washer and classifier. At Central Sand and Gravel plant, Milwaukee, Wis., special washed sand was made by first passing sand through steel cylinder with lifting plates and baffle rings. After scrubbing in this cylinder everything is passed to a twin screw classifier and dewaterer. The washed product is stockpiled by belt conveyor.—EDMUND SHAW

Preliminary tests on effect of size of aggregates on strength of concrete. C. E. PROUDLEY. *Nat. Sand Gravel Bul.*, Dec., 1930, V. 11, No. 12, p. 13-18.—The laboratory of the National Sand and Gravel Association has carried out some preliminary tests in an effort to obtain some definite information on effect of size of aggregates on strength of concrete. One grading of sand and four gradings of gravel were used. The tests were made under the following conditions: The gravels, having different maximum sizes, were graded from fine to coarse in a similar manner; the sands and cement were of constant physical characteristics; and the consistency of the concrete as measured by flow table was same for all aggregates and proportions. While their principal value has been to point out some of requirements for more comprehensive investigation to be conducted in the future, following conclusions may be drawn from a study of results of these preliminary tests: (1) The coarser sizes of gravel gave higher strengths in compression and modulus of rupture than smaller sizes. For the average of proportions studied, the ¾ in. maximum size of gravel was about 12 per cent higher in compression and 7 per cent higher in transverse strength than the ⅜ in. maximum size. The 2 in. maximum size was about 11 per cent higher in compression and 8 per cent in transverse strength than the ¾ in. maximum size. (2) The richer mix, which also contained a higher proportion of sand, was less affected by changes in maximum size of aggregate than the leaner ones. (3) Test results showed fairly uniform relations between strength and water-cement ratio and between strength and void-cement ratio. They indicate that effects of changes in size of coarse aggregate were due to their influence on the quantity of mixing water required to produce uniform workability. (4) Coarse aggregates of different maximum sizes require different quantities of mortar to produce comparable workabilities. Accurate control of this factor is necessary in order to obtain proper comparisons. Further tests with particular attention to this factor are required to permit of more definite conclusions concerning effect of maximum size of coarse aggregate.—P. MCKIM

Dual washing system at new Clarence plant. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 33.—The new plant of the Clarence Sand Co., near Alexandria, New York, has a dual washing and screening system. The excavation is by 3-yd. scraper pulled by a 150 h. p. motor. This discharges to a hopper with a 30-in. belt conveyor feeder. A 24-in. belt conveyor takes the material to a 48-in. by 8-ft. vibrating screen with 2¼-in. mesh screen above and ¼-in. below. The oversize goes to 26-in. jaw crusher; intermediate sizes to double screw washer; the sand to the sump of an 8-in. pump. This pumps at about 80 per cent solids and 20 per cent water through a 6-in. pipe and it takes the place of an installation in which the sand was dewatered by drags and by screws and then carried by a belt conveyor. The washed gravel (with

crusher product added) goes to secondary washing plant above the bins. The gravel is rinsed and sized by vibrating screens and the sand is classified by a classifier with automatic discharge. The fine sands in the classifier overflow are recovered in automatic settling tanks.—EDMUND SHAW

Young men build one crusher stone plant. RAY F. SCHNEIDER. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 48.—At plant of Bussen Quarries, Jefferson Barracks, Miss., mining has superseded quarrying, the overburden being from 10 to 30 ft. Another quarry has been opened to be worked by an open pit. This will be used to supply stone in good weather and mining will be practiced in cold and bad weather. The stone is carried from the quarry face to the plant in 1¾-ton and 5-ton trucks. Entire crushing equipment is one "Jumbo" hammermill, which can reduce large rocks to 2 in. and finer in one operation. Feed opening is 36 x 42 in., fed by apron feeder. Crushed material goes by elevator to revolving screen, making four products. Oversize goes back to the crusher by a conveyor belt and chute. The minus ¾ in. goes to vibrating screen and other products directly to bin. Mixing conveyors below bins permit furnishing any desired combination of sizes.—EDMUND SHAW

Control and improvement of gauging of sand, gravel and river pebbles for concrete. A. JANOUSEK. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—It is the exception for gravel taken from river to correspond with rational requirements for concrete. One of most important objects of control is a granimetric analysis, as accurate as possible, promoting practical and economical improvement of aggregate, and determining what conditions govern strength and other required qualities of concrete. Chief points are: (1) Determination of sizes of stones and grains, their geometric shape, and importance to be attached to them as regards strength of concrete. (2) Determination of individual and total surfaces of grains, and influence of this on strength of concrete. (3) Determination of quantity of grains having equal dimensions, and relative proportions. (4) Application of these results to improve economically gravel as obtained. Usual procedure consists of successive siftings of natural mixture. Another way is to measure dimensions of grains directly, which, however, can be done only to a small quantity. Superficial area of grains can be found only approximately by experimental methods, or by calculation based on results of sifting. To make practical and economical use of these results a continuous control of works is necessary, and parallel with this control, a system of regulating mixture which must operate as automatically as possible and be independent of any initiative on the part of unqualified persons. There is still no satisfactory experimental method for determining exact surface area of stones. Effort has been made to give some indications on the subject of calculating this area, which is of great importance, especially in regard to tensile strength of concrete. Further, the author has examined important question of international unified methods of control, arriving at proposal brought forward in concluding paragraphs of paper; the gist of the matter being a series of screens answering both to scientific and to practical requirements, and suggestion being screens with circular holes .039, .118, .285, .590, 1.180 in., or .039, .079, .197, .394, .787 in. in diameter. Finally, very limited possibilities of using fineness modulus is dealt with and simple method of comparison of granimetric curves and of correcting random gravel is suggested.—EDITIONS LA TECHNIQUE DES TRAVAUX

Slag aggregate follows same course as others. *Rock Products*, Jan. 3, 1931, V. 34, No. 1, p. 144.—Production and sale of crushed slag shrunk 8 to 10 per cent in 1930. Prices were about same as in 1929. Even with present plants operating on 50 per cent capacity basis as indicated, several new plants were built during past year and many older plants were remodeled to operate

more economically. Employes of slag industry worked 75 per cent of time compared to 1929, with an indicated reduction of 25 per cent since 1928. Prices for 1931 are expected to remain at their present level with tonnages showing an increase, especially during the latter half of year. No changes are expected in employes' wages nor will there be an appreciable change in the number of men employed. Important new plants include Buffalo Slag Co., The Duquesne Slag Co., and Cashion Slag Co., the latter notable for simplicity of flow of material through plant. Birmingham Slag Co. is rebuilding its plant at Ensley, Ala., to have capacity of 2,000 lbs. National Slag Association opened a new research laboratory. In the JOURNAL of the Concrete Institute Proceedings, C. A. Hughes and A. S. Levens give some interesting information relative to the use of pulverized slag as a puzzuolanic material. In their tests, two kinds of volcanic ash, a blast furnace slag and burnt shale were used as puzzuolanic material and comparisons of compressive strength, shrinkage, etc., noted. (cf. A. C. I. JOURNAL, Dec., 1930; *Proceedimgs*, V. 27, p. 317). On compression tests in general, slag showed the greatest puzzuolanic activity and greatest strength. Shrinkage increased with increase in effective water to cement plus puzzuolana, slag being an exception to this rule.—EDMUND SHAW

Composition of mixing and application of concrete in engineering works. C. J. JACKAMAN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—This paper, from point of view of a civil engineering contractor, endeavors to record outstanding features which might differentiate concrete works undertaken in England from those in other countries where different conditions prevail. Composition: Aggregates.—More readily available materials for these may be classified as: (a) for southern and southeastern parts of England (with some isolated areas elsewhere), natural river gravels and large areas of suitable sea beach gravels; (b) for districts in the southwestern, western, north midland, and northern parts, various igneous rocks. The former are in great demand and are more readily prepared for use as aggregate and hence in general prove economical; latter class of rocks requiring much more expensive production and preparation have not been so fully exploited. Preparation of aggregates has formed separate and increasingly important branch of concrete industry adjacent to large towns. For works of magnitude, however, every phase of preparation of raw material is commonly undertaken by the contractor on the works site, there being available power-driven machines for various operations of breaking natural rock to acceptable sizes, cleaning, screening and grading. Matrices. —Portland cement is preeminent among all other materials for matrix of concrete work. Improvement of its quality has been so continuous as to have almost entirely out-distanced use of other materials for the purpose. Rapid hardening varieties of portland cement have attained great popularity, justified by advantages they offer in many kinds of work. 1. Proportion of concrete. —For constructional works in England it is still invariable practice of engineer to assume responsibility for specifying proportions of constituent materials and usually maximum and minimum sizes for aggregates and sand. Determination of exact ratios of intermediate grading between limits assigned has not yet been generally adopted. Proportion is still almost invariably stated by measure of volume for coarse and fine aggregate; but for portland cement greater exactitude is sought by proportioning by weight. Little progress has been made in adoption of any method for relating proportion of mixing water to quantity of cement. Due check on quantity of water used is, however, maintained by increasing adoption of slump test of concrete after mixing. 2. Mixing concrete.—Various types of mechanical concrete mixers in general use are noted. "Batch" or non-continuous mixer, is most generally favored for securing uniformity of quality in product. Internal-combustion engines with petrol fuel have outstripped all competitors for motive power for these machines. 3. Distribution of concrete.—English engineers have shown great reserve in permitting use of automatic distributing plant and this attitude is receiving

support from importance now assigned to concrete having minimum water content consistent with easy manipulation. 4. Deposition of concrete.—No generally applicable, economical, and equally satisfactory method of consolidating concrete, when placed, has yet been produced to compare with carefully supervised manual effort. 5. Formwork.—Use of timber for formwork has not yet been challenged by any serious rival, owing to high initial cost of steel forms which forbids their adoption on any but large expansions of repetition work.—EDITIONS LA TECHNIQUE DES TRAVAUX

CEMENT

Studies on mixed portland cements—Part 6. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 279-85. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1191.—Neo-Soliditit cement was made from slate mixtures. Soluble portion of slate was determined by successive treatments with sodium hydroxide solution (10 per cent) and hydrochloric acid (5 per cent). Mixtures of 55 to 85 per cent clinker and 15 to 45 per cent slate gave surprisingly high strength data, higher than pure cements.—A. E. BEITLICH

Studies on mixed portland cements—Part 7. SHOICHIRO NAGAI. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental binding, No. 6, p. 192-6B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Expansion and contraction of several kinds of cements: portland, blast furnace slag, high alumina and soliditit cements (to which certain colloidal materials were mixed in order to correspond with Japanese Neo-Soliditit) were studied. Specimens were heated from 392° to 1472° F. Great shrinkage led to complete destruction of several cement samples. High alumina cement showed greatest resistance. Portland cement shows a considerable increase of free lime content under this treatment. Same increase can be observed between 752° and 1112° F. in the case of soliditit cement and blast furnace slag cement, while free lime content drops sharply when heated from 1472° to 1832° F. A combination of free lime with acid constituents of colloidal materials seems to take place at such high temperatures. (cf. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, p. 279-85.)—A. E. BEITLICH

Special cements and concretes. A. HASCH. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Maximum properties of cement and concrete can be considered from the general physical, mechanical and chemical points of view. The comparison of high early strength portland cement with Ciment Fondu (aluminous cement) shows that the latter is principally used on account of its greater capacity of resistance, as the high early strength of portland and cool setting cements is sufficient for majority of constructional requirements. Also they can be used for concreting at low temperatures. Statistical consideration of experimental results using Bernouilli law of the greatest number appears also to be an important method in future work of material testing.—EDITIONS LA TECHNIQUE DES TRAVAUX

Further developments in closed circuit dry grinding. H. G. WRIGHT. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 46.—Micro-photographs show that products made by reverse current air separation are more angular and therefore have more surface exposed than unclassified products. Hence they should burn more easily. Air classified material is shown to carry less oversize. That which makes good burning in kilns is not amount reduced below 200-mesh but that which is below 100-mesh. Tables are given comparing discharge of tube mills by gravity and air classified reverse current system. These show a less amount retained on 100-mesh for air classified products for pro-

portionately same grinding through 200-mesh. (cf. *Rock Products*, July 19, 1930, V. 33, No. 15.)—EDMUND SHAW

Examination of refractory materials for cement kilns. J. SCHAEFER. *Zement* (Germany), Nov. 20, 1930, V. 19, No. 47, p. 1112-3.—Durability of brick lining of rotary kilns depends largely on structure of bricks (shape and amount of voids and gradation), viscosity and chemical composition of clinker, reaction temperature, composition of bricks and duration of reaction. An improved method is described for testing of bricks under conditions which are similar to those encountered in actual mill practice. Bricks are heated in a high temperature furnace to different test temperatures and a powerful gas flame carrying raw mixture is blown against them. Effect of heat and reaction of clinker particles on brick surface can be easily determined. Also effect of great temperature changes on brick can be studied by rapid removal of brick from furnace. Chemical analysis of top layer shows depth of penetration and changes in composition. Photographs show different behavior of ordinary refractory brick, Dynamidon brick and Magnesidon brick.—A. E. BEITLICH

The grinding of cement and porosity of concrete. A. POULSON. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—No concrete is solid, and to secure greatest protection against chemical action (including that due to infiltration of pure water) a puzzolanic cement must be used to neutralize deleterious lime which will then unite with hydraulic constituents of puzzolana to form compounds like those in cement itself. In addition the "hydraulic index" should be tested and ought not to be less than $\frac{7}{8}$; also mechanical strength. Reference is made to previous publications in respect of rational use of puzzolanas, and of their examination: 1912: New York Congress on Materials: *Cement in Seawater*. 1923: London Congress on Navigation; No. 53: *Portland cement in seawater and chemical value of puzzolanas*. 1924: Cairo Congress on Navigation: *Hydraulic mortar in seawater*. 1929: For Zurich Congress on Materials, 1931: *Compactness and chemical resistance of concrete*.—EDITIONS LA TECHNIQUE DES TRAVAUX

Fused cement and its influence in industry. EMIL STOCK. *Osterr. Bauzeitung* (Austria), Nov. 15, 1930, V. 6, No. 46, p. 752.—Resume in German of Italian article dealing with development of high alumina cement industry. Fused cements are used with advantage (1) for jobs which must be finished in a short time, (2) for winter construction and (3) for structures which are exposed to chemical actions or aggressive solutions. Rapid development of this type of cement led to increased efforts to improve ordinary portland cement and to study its physical and chemical properties on a scientific basis. (cf. *L'Industria Italiana del Cemento* (Italy), 1930, No. 5.)—A. E. BEITLICH

Researches on the rotary kiln in cement manufacture—Part 9. GEOFFREY MARTIN. *Rock Products*, Dec. 6, 1930, V. 33, No. 25, p. 54.—Heat required to decompose $CaCO_3$ at different temperatures is determined. This is usually taken to be 774 B. t. u. per lb. at ordinary temperatures. But calcite, raw material of cement, takes 756 B. t. u. per lb. From researches of several investigators, especially John Johnstone, a formula is worked out and steps are given in detail. From formula a table has been made giving the heat of disassociation of $CaCo_3$ in the rotary kiln. This shows that, beginning at ordinary temperatures, required heat is about 756 B. t. u. for calcite, but at 1443° F., (the lower limit of decomposition in rotary kilns) it is 685 B. t. u. At 1481° F. (upper limit of initial decomposition), it is 681.6 B. t. u. With increasing temperatures B. t. u. required decrease, dropping from 644.2 at 1832° F. to 341.1 at 3632° F. At 4940° F. no heat is required but there is evolution of 1.25 B. t. u. At 5432° F. there would be an evolution of 156.1 B. t. u. It would be impossible to secure this because of enormous pressure that would be required to keep $CaCO_3$ from splitting into CaO and CO_2 before reaching critical temperature of 4940° F.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture—Part 10. GEOFFREY MARTIN. *Rock Products*, Dec. 20, 1930, V. 33, No. 26, p. 49.—In calculating amount of heat absorbed in making one pound of portland cement clinker, author assumes clinker containing 64 per cent CaO, 7 per cent Al_2O_3, 3 per cent Fe_2O_3, 22 per cent SiO_2 and 4 per cent non-essentials. To make 100 lb. there will be required 119.5 lb. $CaCO_3$, 18.46 lb. kaolin, 15.40 lb. hydrated silica, and 3.12 lb. ferric oxide. A study is made of each of these showing quantities of heat required to dehydrate, when this is necessary, and to raise material to temperature required to make clinker. The final quantity of heat required for 100 lb. clinker figures to 81,192 B. t. u. for decomposing $CaCO_3$, less 17,993 B. t. u. evolved in exothermic reactions, plus 28,665 B. t. u. absorbed in raising 100 lb. of clinker from 1481° F. to 2498° F., a net total being 91,864 B. t. u. Conclusion is that it should be carefully noted that it is fallacy to state total minimum quantity of heat required to make 1 lb. of clinker is 918.6 ÷ 652.5 = 1571.1 B. t. u., because this statement takes no account of quantity of heat recoverable from hot clinker and evolved gases. In making 1 lb. clinker raw material mixture must be subjected to steadily increasing temperature starting at 60° F. and finishing at about 2,498° F., and raw mixture must absorb 652.5 B. t. u. below 1,481° F. and 918.6 B. t. u. above 1,481° F. before 1 lb. clinker can be formed.—EDMUND SHAW

Studies on calcium ferrites and iron cements—Part 3 and 4. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding No. 6 and 7, p. 190-2B, 256-9B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1191.—New data are presented of this comprehensive investigation (cf. *Kogyo Kwagaku Zasshi* (Japan), May, 1930, V. 33, Supplemental Binding No. 5, p. 161-4; JOURN. Amer. Concr. Inst., Dec., 1930, V. 2, No. 4, *Abstracts*, p. 96). Dicalcium ferrite, $2CaO.Fe_2O_3$, reacts with water in formation of calcium hydroxide which produces its strength properties. These decrease in water storage and stay fairly constant in air storage. Constancy of volume of dicalcium ferrite mortar was studied by Bauschinger and Le Chatelier methods and compared with properties of portland cement and Kuhl cement. When mixtures of SiO_2, Fe_2O_3 and CaO are fused between 2552° and 2732° F., calcium ferrite is formed first, then combination of CaO and SiO_2 to di- and tricalcium silicate takes place. Synthetic mixtures in the field SiO_2—Al_2O_3—Fe_2O_3—CaO were prepared by replacing 1 part of ferric oxide (8-9 per cent) by alumina (2-3 per cent) at temperatures between 2552° and 2732° F. Cements consisted of about 15 to 16 per cent dicalcium ferrite, 6 to 7 per cent tricalcium aluminate and various amounts of di- and tricalcium silicate. Formation of tricalcium silicate was only possible, when ferrite or aluminate were present.—A. E. BEITLICH

Studies on calcium ferrites and iron cements—Part 5. SHOICHIRO NAGAI AND KATSUHIKO ASAOKA. *Kogyo Kwagaku Zasshi, J. Soc. Chem. Ind.* (Japan), 1930, V. 33, Supplemental Binding, No. 8, p. 312-5B. Reviewed in *Zement* (Germany), Dec. 11, 1930, V. 19, No. 50, p. 1192.—Temperature of combination of iron cements lies above sintering temperature of ordinary portland cement due to small amount of calcium aluminates. These temperatures are between 2642° and 2732° F. Chemical composition is about 65.2 per cent CaO, 23.3 per cent SiO_2, 2.50 per cent Al_2O_3 and 9.00 per cent Fe_2O_3 and their constitution is approximately 52.2-54.5 per cent tricalcium silicate, 20.2-23.7 per cent dicalcium silicate, 3.5-5.8 per cent tricalcium aluminate and 14.8-16.0 per cent dicalcium ferrite. Molecular ratio between CaO and SiO_2 is between 2.62 and 2.67. Strength data obtained by small-piece testing methods are lower than ordinary portland cement strength.—A. E. BEITLICH

PROPERTIES OF CONCRETE

The ideal gauging of concrete mixes. H. IMBERECHTS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The object of the researches dealt with in this paper is to establish a method of calculating proportions for concrete mixes based upon knowledge of curves of granimetric composition for elements involved. Investigation starts with the Fuller-Bolomey curve, $P = A + (100 - A)\sqrt{d/D}$. After determining best value for constant A, percentage of fine elements, including cement, not exceeding 0.02 in. size, which corresponds to the strongest concrete—for a given water content—still easily workable and of satisfactory appearance when fractured, researches were extended to cover all acceptable mixtures between two limits. The lower limit was introduced to secure concrete still easily workable into place, the upper limit to restrict falling off in strength. Reason for laying down these limits is impossibility of making particular cases conform precisely to a given "typical composition"; it is necessary to allow certain tolerances, and curves of "limiting composition" furnish tolerances which can be prescribed.—EDITIONS LA TECHNIQUE DES TRAVAUX

Control of concrete construction of the German railroad concern (Reichsbahn). VOGELER. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 236-61.—After discussing contents of recently issued German specifications for mortars and concrete, author describes field control outfit and equipment for stationary concrete control laboratories. All materials: cement, lime, trass, aggregates and mixing water are submitted to careful examination. Concrete is tested for its tensile, compressive and flexural strength, water permeability and resistance against aggressive solutions. Furthermore, admixtures, water proofing materials, protective coatings and bricks are examined. Detailed description is given of equipment and machinery for strength and other tests. Special consideration is given to sieve analysis and exact proportioning of aggregate gradings. Samples of good concrete and concrete that failed, with necessary explanations facilitate understanding of specifications. Progress during first year is very satisfying. Program for second year includes examination of methods and materials for water tightening of concrete foundations, protective coatings, plasters and surface finishing. These will be studied especially on abutments, supporting walls, piers, bridges, culverts and tunnels.—A. E. BEITLICH

Deterioration of concrete in hydraulic structures. A. EKWALL. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Inspections of hydraulic structures in Sweden displayed some defects in concrete 10 years old. Continued observations proved, that concrete in some cases was subject to increasing deterioration, apparently due to qualities of concrete materials, to methods of making concrete or to conditions of exposure. Investigations above described were arranged to ascertain causes of such increasing deterioration. It had been observed that deterioration occurred only in structures exposed to water on one side and was caused by solubility of lime and decomposition of other chemical combinations in hydrated cement. Particular attention was therefore placed on solubility of cement and aggressiveness of water. In combination with investigation relating to these factors, practical experiments have been made with concrete exposed to low water pressure for several years. Investigation proved that Swedish natural waters need not be considered aggressive in this respect. Importance of using sand, relatively free from organic substances, was impressively established. Leaner mixtures than 1:4½ should not be used for concrete exposed to water pressure from one side. Replacing cement by an admixture of 10 per cent lime or 5 per cent calcium chloride improved watertightness,

while 25 per cent trass or slate powder gave practically no improvement.—EDITIONS LA TECHNIQUE DES TEAVAUX

Investigation of the behaviour of cement mortar in hot water. "Versuche ueber das verhalten von zementmoertel in heissem wasser." OTTO GRAF. Report of the Institute for Testing Materials at the Technische Hochschule in Stuttgart (Germany), during the years 1928-1929; Publication No. 62 of German Committee for reinforced concrete. Berlin 1930, Wilhelm Ernst und Sohn, R. M. 4.30. Reviewed in *Bautechnik* (Germany), Dec. 5, 1930, V. 8, No. 52, p. 770.—Test pieces of portland cement, high early strength cement and high alumina cement were exposed to water of 68, 122 and 194° F. Permanent storage of testpieces in hot water (194° F.) resulted in an increase in strength of lean mixtures (1:6) while rich mixtures (1:3) lost some of their strength. Behavior in water of 122° F. was varying according to different kinds of cement. Frequent rapid changes of temperature led to considerable reductions in strength and were different with different cements. Conclusions are drawn concerning use of such cements for special structures (hot water tanks).—A. E. BEITLICH

The effect of temperature on flat arches constructed of reinforced concrete. CHARLES S. GRAY. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—First part studies temperature range which may be expected in an arch, variation of temperature throughout the bridge and ratio of internal to external temperature. Second part treats three hinged arches and considers movement of crown hinge and change in horizontal thrust for a temperature variation of 30° F. Third division considers arches of two hinges; curves and tables are given showing variation of temperature thrust and temperature moment at all points of span for various ratios of rise to span for any temperature range. In fourth section are treated arches without hinges; curves are given to show variation of temperature thrust and temperature moment at all points in span for various ratios of rise to span. Fifth section considers effect of change of shape of arch axis upon temperature thrusts and moments.—EDITIONS LA TECHNIQUE DES TRAVAUX

Experiments with columns reinforced with steels of high strength properties. R. SALIGER. *Osterr. Bauzeitung* (Austria), Nov. 1, 1930, V. 6, No. 44, p. 723-30.—Progress of joint investigation of a German and an Austrian institute for testing materials on the subject of reinforcing of concrete columns is reported. Tests with steels of high quality are finished and results reported. Eight different types of columns with octagonal cross-section and circumferential reinforcements of 0.78-in. hoop irons or 0.2-in. round steel were broken. Longitudinal reinforcements consist of 4, 6 or 8 steel rods, 1.18 in. and 1.65 in. thick. Diameter of columns is 13.8 in., their length is 4 and 9.8 ft. Tests in Vienna were made with columns, reinforcements of which were welded in center. Tables show changes in form of columns when exposed to pressures from 10 to 200 tons. Under high pressures, uniformity between column core and outer layer is destroyed. Columns of 9.8 ft. length show a 15 per cent lower compressive strength than 4-ft. columns. Columns with welded reinforcements are of no advantage. Results of tests are carefully analyzed and conclusions are drawn for application in practice.—A. E. BEITLICH

Changes in the action and strength of hyperstatic reinforced concrete structures with time. H. LOSSIER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Concrete undergoes with time a number of changes modifying action of reinforced concrete structures both from point of view of elasticity and of strength. The former changes, in structures which are hyperstatic, affect sharing loads between the different members. Although generally speaking it does not appear

a priori that these changes are of such nature as to affect safety of the works, yet it is important that question should be studied and checked by experiments in hope of arriving at a solution.—EDITIONS LA TECHNIQUE DES TRAVAUX

Measurement of temperature in concrete during its hardening. F. G. PEREZ. *Ingenieria* (Mexico), Nov., 1930, V. 4, No. 11, p. 418-420.—Small cracks in the buttresses of the Rodriquez Dam (Ambursen Type) in lower California were investigated by a group of consulting engineers who concluded that temperature developed in interior of mass during hardening was one of the causes. This led to investigation of effect of fineness of cement on temperature produced, the data from which is shown graphically in the report. The investigation compared a cement ground to 94 to 96 per cent passing a 200-mesh sieve with one that had 84 to 86 per cent passing that sieve. Temperature measurements were made electrically with resistance type pyrometers inserted in the concrete at five points. Readings were taken at short intervals during first day and at increasing intervals thereafter until they became practically constant. Temperature during setting of concrete made of finely ground cement reached 78° F. while air was 67° F. Concrete made of more coarsely ground cement reached 71° F. while air was 53° F. Temperature of concrete ascended for three days for finely ground and four days for coarsely ground cement. Maximum temperature attained in the concrete with finely ground cement was 126° F. and 118° F. with the coarsely ground cement. Both kinds of cement maintained a temperature higher than surrounding air for two weeks. Maximum temperatures were obtained within four ft. of the exposed side of concrete.—C. G. CLAIR AND M. N. CLAIR

Report on the Swedish investigations and experiences concerning shrinkage and temperature in concrete. N. ROYEN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Great variation in temperature (95° F. to —56° F.) existing in some parts of Sweden have made it necessary to pay particular attention to such factors as heat generated by setting and hardening of concrete and the influence of temperature and shrinkage. Investigations have been made to find relation between temperature of concrete and air temperature for different slab thicknesses. Heat generated during setting has been investigated by experiments for a number of different brands of cement. Amount of high alumina cement required to obtain, by heat generation, a sufficiently high temperature for curing concrete made at temperatures below 32° F. and effect of low temperature on hardened concrete have been objects of other investigations. Results of experiments on shrinkage and swelling of concrete are given, and finally are stated some measures taken in design and construction of dams in northern Sweden to decrease effect of temperature and shrinkage.—EDITIONS LA TECHNIQUE DES TRAVAUX

The consistency of cast stone. H. A. HOLT. *Conc. Building Conc. Products* (England), Jan. 1931, V. 6, No. 1, p. 2.—The difficulty of measuring consistency is pointed out, since slump tests are for use with much wetter mixes than are encountered in the manufacture of cast stone. "Dry" refers to a mortar which will ball in the hand and leave it clean, but will break and crumble when dropped. It may be mechanically tamped in a mold without the appearance of moisture on the surface, and without becoming plastic. "Semi-dry" mortars will ball in the hand and leave it clean, but when thoroughly tamped in a mold, will become just plastic but no more, and water can just be seen on surface. "Plastic" mortar will just moisten the hand and will ball, barely fails to slump, and will not flow down an inclined plane, but when tamped becomes a stiff quaky mass with some moisture on the surface. For strongest, densest and most impermeable concrete minimum amount of water required to produce plasticity only, must be used. In case of 1:2:4 concrete, 6 per cent water by weight of total dry mixed materials would thus be required.

In many methods of manufacture of cast stone, wet mixes are unavoidable. In case of spun concrete moisture is brought to surface by centrifugal action and is removed before concrete has hardened. On account of excessive labor cost, dry mixes are only practicable with mechanical tamping or consolidation. When little mixing water is used proper curing is essential for case hardening the concrete, and completing hydration of cement on the surface. Experiments show that maximum temperature accompanied by saturated humidity produces maximum strength in minimum time. For surfaces which are to be tooled or brought to an ashlar finish, fine sand and semi-dry or dry mixes are recommended. Wet mixes give better results when coarser aggregates are used, and polished or scrubbed surfaces are required. Plastic concrete tends to produce less crazing and greater adhesion of cement to reinforcement, less risk of disintegration by frost or acid atmospheres than dry and semi-dry mixes, but is less speedy in production.—JOHN E. ADAMS

The grinding of cement and porosity of concrete. (See MATERIALS—CEMENT)

ENGINEERING DESIGN

BUILDINGS

New construction of a recovery home for children in Koenigsfeld (Germany). ALFRED FISCHER. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1092-3.—Building is made as homogeneous reinforced concrete skeleton structure and satisfies all demands of modern hygiene. Cantilevered construction of second floor forms roof above dry, open air playground and is supported by reinforced concrete columns.—A. E. BEITLICH

Progress of experimental statics of rigid frames. F. KANN. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Method developed by author for experimental solution of frames with a high degree of rigidity rests on direct measurement of rotation of joints and bars of a system elastically deformed. In this purpose he has made use of a novel manner of constructing model of bars of celluloid joined at corners by blocks of hard wood which permits of production of very considerable deformations. By design of elastic member and measurement of angles of tangents and chords we can measure exactly rotations, which allows of calculation of moments at joints or their ratios useful for completing or controlling the calculations. Comparison has been made for frames without struts at angles and for frames with struts reaching to 1/5 of length of beams and uprights. Measured angles agreed closely with calculations when taking action of struts into account. Use and development of method in practice for systems with a high degree of rigidity is therefore very promising.—EDITIONS LA TECHNIQUE DES TRAVAUX

Collapses of reinforced concrete structures. LUIGI SANTARELLA. *Osterr. Bauzeitung* (Austria), Nov. 22, 1930, V. 6, No. 47, p. 764—Reasons for collapses of reinforced concrete structures are classified in 4 groups: (1) errors in calculation, design or construction, (2) impracticable construction or erroneous reading of blueprints, (3) bad condition of structural materials or incorrect mixtures and (4) too early removal of forms and lack of protection against unexpected damages. Directions to remedy such faults are given. (cf. *L'industria Italiana del Cemento* (Italy), June 1930.)—A. E. BEITLICH

Tests to destruction of a reinforced concrete floor. L. VANDEPERRE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Tests described had two objects: (1) determination

of factors of safety—with regard to initial cracking of concrete—exceeding limits of elasticity of steel and final rupture of a complete structure constructed of reinforced concrete. Calculated on bases of hypotheses commonly accepted in resistance of materials and stability of constructions. This complete structure was a normal floor of slabs and lower ribs of 20 ft. and 25 ft. made in the yard, without particular care or special supervision, by workmen usually employed on similar work. 2. Determination of same dimension comparable with first for calculated permissible loads, and made under same conditions but of which organic conception is based on use of concrete in very thin curved slabs stiffened by their curvature. Tests consisted of loading to breaking point two floors and measuring deflection of principal beams. Preliminary calculations, description of test loads and measurement of deflections are reviewed with description of tests with photographs and diagrams of cracks and of rupture. Tables and diagrams review results of tests and other tables review comparative conclusion as to resistance and cost of two systems tested. In conclusion the following points are cited: (1) Regularity of mixing without particular care. Special supervision has been within about 15 per cent. (2) The factor of safety against initial cracking in normal floor has been less than 1.0 for one of the beams. A mean value would be about 1.2. But unfavorable circumstance as concreting in very dry and warm weather may have made it fall below 1.0. For the special slab it was equal to about 1.17. (3) The factor of safety to exceeding the limits of elasticity of the steel was 2.4 for the normal floor and 3.0 for the special floor. (4) Factor of safety against rupture was greater than 3.6 for normal floor which could not be loaded to destruction but which would have certainly failed by drawing of the steel. It was equal to 3.49 for special floor which failed prematurely by crushing of concrete in compressed portion of marginal beams, a central drum doubly reinforced would have rendered the section of rupture rigid transversely and thus increased security of the whole. (5) All failures by bending by crushing of concrete have occurred by stepping along planes oblique to compressive stress. (6) Refacings or renderings have behaved perfectly up to point of rupture, these inevitable repairs serve then in practice to perfect work from point of view both of resistance and appearance, and it is therefore necessary to be doubly careful in this respect and not to treat them as mere added veneer.—EDITIONS LA TECHNIQUE DES TRAVAUX

Reinforced concrete in Great Britain. A. KIRKWOOD-DODDS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, 1930.—First reinforced concrete structure designed by Hennebique was erected in Belgium nearly 50 years ago and some noteworthy examples of his early work are to be found in the city of Liege today. Having been convinced by practical experience of possibilities of concrete in combination with steel, Hennebique formulated a complete method of design and in 1892 established his well-known headquarters in Paris, whence the Hennebique system was spread throughout Europe and to almost every part of the world. Great Britain was not far behind France and Belgium in taking up the new method of construction, for in 1896, the late L. G. Mouchel became the pioneer of this form of construction in that country. Much effort was necessary before reinforced concrete was accepted by British engineers and architects, but in 1898 its claims were recognized by the chief engineer to the Mersey Docks and Harbor Board and by the Great Western Railway engineers, and a little later, Mr. Mouchel was appointed consulting engineer for New General Post Office, London. At time of Mr. Mouchel's death in 1909, some 800 reinforced concrete structures had been completed in the United Kingdom. In referring briefly to methods of calculation, it is pointed out that several structures described were designed by means of formulae evolved by Hennebique some 40 years ago, and ever since then used with consistently satisfactory results, a record attributed to the fact that the formulae and the essential factors used in their practical application were derived from scientific tests and actual

experience. Some typical examples of reinforced concrete building construction on a large scale in Great Britain, referred to are: General Post Office Buildings, London.—Comprising two blocks connected by a two-story basement, these buildings have a total volume of 8,500,000 cubic feet. They were designed on the monolithic skeleton principle, all loads, including that of architectural masonry, being carried by reinforced concrete construction. Buildings embody interesting structural details and use of reinforced concrete resulted in a very large reduction of capital expenditure. Royal Liver Building.—This conspicuous landmark on quayside in Liverpool rises 360 ft. above foundation, and is 301 ft. by 177 ft. in plan. Masonry enclosing walls are carried entirely by a monolithic reinforced concrete skeleton. Long span beams, interior arches and main columns, carrying loads up to 1,500 tons each are some features of building. Government Stationery Office, London.—An impressive example of reinforced concrete work, including two eight-story buildings, one 106 ft. by 80 ft. and other 480 ft. by 215 ft., connected by a three-story bridge. Wall panels of skeletons are mainly occupied by windows, providing admission of natural light. Railway Station and Warehouse, Newcastle-on-Tyne.—Building, 430 ft. by 178 ft. in plan by 83 ft. high, comprising high and low level railway stations, with six lines of rails, above them being a three-story warehouse and a flour storage of 500,000 cu. ft. capacity with sloping floors supported by reinforced concrete lattice girders carrying loads up to 300 tons each. After over 25 years of heavy continuous duty, structure still justifies Hennebique methods employed in its design. Dock Warehouses and Grain Silos.—The first example is a range of five transit sheds at Manchester docks, with frontage of 2250 ft. and depth of 112 ft. from front to back, total floor and roof area being nearly 22 acres. Unusual feature is railway tunnel passing obliquely through one of buildings. Among other grain silo buildings in Manchester docks is one with a storage capacity of 1,680,000 bu., the largest structure of its class in Great Britain. Another interesting example is group of six cylindrical silos near Newcastle-on-Tyne, each compartment being 72 ft. high by 45 ft. in diameter, embodied in a structure 176 ft. by 92 ft., 109 ft. high.—EDITIONS LA TECHNIQUE DES TRAVAUX

BRIDGES

Pyke's Creek bridge. GEOFFREY OWEN THOMAS. *J. Inst. Eng.* (Australia), Oct., 1930, V. 2, No. 10, p. 369.—Paper deals with choice, design and construction, and testing of a 5-span continuous reinforced concrete road bridge, 246 ft. long, over a reservoir near Bacchus Marsh, Victoria. With such long spans, considerable economy is obtained by using beams of widely varying moment of inertia, the deep haunches greatly reducing the positive moment in mid-span. Temperature and secondary stresses are investigated. Erection was simplified by incorporating, as reinforcement, in the superstructure, a light welded steel truss, from which forms were supported. Deflection tests were found to agree with calculated values if E is taken as 6,000,000 lb. per sq. in. The average compressive strength of test cylinders was 3770 lb. per sq. in. Tests proved the waste in adhering to value of E equal to 2,000,000 in cases where concrete is properly supervised in construction. Detailed calculations are included.—JOHN E. ADAMS

Working stress of 1,200 lb. used for 39 ft. concrete slabs. CLARK H. ELDRIDGE. *Eng. News Record*, Dec. 18, 1930, V. 105, No. 25, p. 981-982.—Reinforced-concrete slab construction on 39-ft. spans with concrete working stresses of 1,200 lb. per sq. in. was required by the limited head clearance at the Second Avenue extension viaduct in Seattle. Construction includes use of several unusual details, including an unsupported expansion joint through a 15-in. slab, using steel dowels and a novel method of supporting slab on lower flange of a plate girder. Live load was taken as two 20-ton trucks with allowance of 100 per cent on rear axle of one truck. Working stress in steel was taken at

16,000 lb. per sq. in. Under these conditions slab thicknesses for four spans were 12, 13, 15 and 12 in. respectively. Main reinforcement consisted of 1⅛-in. round bars at 4-in. centers for the 36 and 39-ft. spans, with ½-in. round spacer bars at 4 ft. intervals. Concrete was designed for 3,000 lb. per sq. in. and a fiber stress of 0.4 of this value or 1,200 lb. was adopted. Strength of concrete in structure, as shown by 27 test cylinders broken at 28 days, averaged 3,559 lb. per sq. in. with a high strength of 4,760 lb. and a low strength of 2,380 lb. Two expansion joints were provided about 60 ft. apart at right angles to supporting bents. These joints necessitated some method of distributing heavy wheel loads between adjacent slabs. This was done by encasing the edges of the slabs in steel channel sections, anchored into edge of concrete by means of steel straps, and using heavy steel dowels to transmit the load across the joint. Backs of channels are spaced 2½ in. apart to allow for expansion.—D. E. LARSON

Design of arched bridges with fixed supports. H. CARPENTER. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 11-12, p. 612-622, p. 673-685.—Method is given of obtaining moments and stresses due to live load, and is based upon assumption that uniformly distributed live load can be used in place of usual procedure of using influence line. Investigations were restricted to critical sections at crown and springing. From study of properties of influence line maximum positive moment at crown is obtained by loading central part of the arch. Values of moments, vertical reactions at supports and horizontal thrust are obtained using influence line by superimposing unit loads at most unfavorable positions. Maximum values of moments, vertical reactions and horizontal thrusts are thus obtained. However, moment = αwl^2, horizontal thrust = $\beta wl^2/r$ and vertical reaction = γwl, where w = the unit load per ft. and α, β and γ are coefficients. By equating the results obtained by use of the influence lines to these expressions corresponding values of coefficients, α, β and γ are obtained. Values of these coefficients are given for different rise-span ratios and for different ratios of depth at crown to depth at springing. Tables of coefficients are given for parabolic, open spandrel and filled spandrel arches. It is necessary to know values of uniform load w in order to use above coefficients to obtain moments, thrusts and reactions. Values of equivalent uniform load w were computed by use of influence line and actual wheel loads. Equating values thus found to values in terms of w, values of w were obtained. Results of these investigations are plotted giving values of equivalent uniform load w for spans varying from 25 to 400 ft. Curves are plotted for negative and positive moments at springing, positive moment at crown, horizontal thrust at crown and springing, and vertical reaction at springing. Study was made to determine the relation between spacing of longitudinal beams and resultant load imposed upon arch ring. A point of interest from a study of tables of coefficients for moments is that as the ratio of the springing thickness increases, arch axis remaining the same, moments at springing increase, but moment at the crown decreases. The result is a reduction of moment at center and a corresponding increase of moment at support. Application is made of the above method to an open spandrel arch of 150 ft. span, rise of 22 ft. 6 in., depth at crown of 2 ft. 3 in. and depth at springing of 3 ft. 4½ in. Stresses are computed and a comparison made between the approximate method presented and the "exact" method showing close agreement. Value of the horizontal thrust by the "exact" method is 78,000 lb. and by above approximate method is 82,200 lbs.—JOSEPH MARIN

DAMS

New type of arched dam. A. PENA BOEUF. *Summary Reports*, First International Congress for Concrete and Reinforced, Concrete Liege, 1930.—Serious difficulty in calculation of arched dams, in which certain hypotheses must be made to consider relative influence of jutting in the foundation and

the side walls, is radically corrected in type of dam proposed, and in which in addition to suppression of indeterminate factor of elasticity, there is an enormous saving in amount of materials used. Fundamental principle is the division into horizontal isostatic arches and the ease with which the calculation of elasticity can be applied to them.—EDITIONS LA TECHNIQUE DES TRAVAUX

Stresses in dams. S. CAROTHERS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The usual graphic method of obtaining stresses in dams necessitates beginning at the top and working downwards towards the base. It will thus be seen that any error in earlier part of work will be perpetuated to the end and will vitiate results obtained for lower part of dam. Paper gives a method of obtaining stresses at base by a direct process of calculation and this should prove useful as a check on stresses as found by graphical methods. In addition, the paper introduces polar coordinates which have great advantage that for dams of approximately triangular section the principal stresses are obtained direct for the front and flank of the dam. These are points where stresses have maximum values. Stresses are obtained from Airy's stress function x by methods fully explained in Love's Elasticity.—EDITIONS LA TECHNIQUE DES TRAVAUX

MISCELLANEOUS

Calculation of secondary effects in composite girders. A. PENA BOEUF. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Considering these girders, for purpose of calculation, as an articulate isostatic system, to each diagonal one applies the corresponding tensile stress and when these diagonals are virtually suppressed, the girder is calculated as a Vierendeel girder. To the law of moments resulting at each joint, the distribution is applied to the parts which meet thereon, in accordance with the relative flexibility of each.—EDITIONS LA TECHNIQUE DES TRAVAUX

Reports on new experiments concerning reinforcing of concrete beams against thrust stresses. OTTO GRAF. *Report 33rd main meeting German Concrete Association*, March 17, 1930, p. 137-42.—Results of three series of tests on reinforced concrete beams are reported; special consideration was given to reinforcing against thrust stresses. Several weak points in German specifications for reinforced concrete of 1925 are discussed and certain changes are proposed.—A. E. BEITLICH

Tests on columns of bound concrete reinforced with bars of special steel and welded. R. SALIGER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—1. Tests deal with columns of bound concrete with a high percentage of reinforcement of special steel. 2. Load carried results in every case from the sum of the resistance to compression of concrete core and longitudinal reinforcement as well as from resistance to extension of binding. 3. The share of resistance of concrete amounts to $\frac{1}{3}$ to $\frac{1}{6}$ according to importance of reinforcement, which reinforcement supports from $\frac{2}{3}$ to $\frac{5}{6}$ of the total load. 4. Welded longitudinal reinforcement has a resistance to compression 10 per cent inferior to that of non-welded bars. 5. On account of their high resistance to compression the section of such columns should not occupy more space in the structure than columns of rivetted steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Design of circular tank of reinforced concrete. I. L. RASMUSSEN. *Eng. News Record*, Dec. 11, 1930, V. 105, No. 24, p. 923-925.—The designer of circular tanks of reinforced concrete has to overcome theoretical obstacles not generally met with in design of other structures. The walls of the tank are

rigidly fixed to the base, which, for purpose of analysis, may be considered immobile. The radii of the cylindrical sides tend to lengthen under pressure. At base, the walls are completely restrained and their radii cannot change in length. As distance from base to any horizontal section of wall increases, the unrestrained movement of walls also increases until a point is reached where movement is entirely unrestrained. To determine this distance and the moments and stresses which are set up between the base and this point through the restraint of the base is chief problem to be solved in design of circular tanks. The author presents a simple and direct mathematical analysis for determining these moments and stresses.—D. E. LARSON

Articulated concrete revetment construction on the Mississippi River. MORRIS W. GILLAND. *Eng. News Record*, Dec. 25, 1930, V. 105, No. 26, p. 996-1003.—Early in the task of regulating the Mississippi River for navigation and flood protection, it was manifest that some of its banks must be prevented from caving. Gradually brush mattress superseded all other types of revetment structure until 1915 when its position began to be contested by concrete. Concrete was the first thought because it had served well for upper bank paving, was even more durable under water and suffered little from being alternately wet and dry. The articulated-mat revetment being used at present consists of a series of lapped mats extending from waters edge to bottom of underwater bank and outshore on river bottom some 50 ft. farther. Each mat is 140 ft. wide alongshore and from 300 to 350 ft. long outshore from the water's edge. Adjacent mats overlap 10 ft. Each mat is made up of sections, 4 by 25 ft. strung on steel cables, which extend upbank and are anchored inshore. Construction consists of four operations: (1) fabrication of articulated mat sections; (2) grading upper bank to 1 on 4 slope; (3) assembling and sinking sections to make continuous revetment; and (4) paving sloped bank with concrete. Each of four operations requires separate plant (c. f. A. C. I. JOURNAL, June, 1930, p. 799, *Proceedings* V. 26, 1930).—D. E. LARSON

Nomogram for calculating average diagonal tension in a given section of beam. JORGE QUIJANO. *Ingenieria* (Mexico), Nov., 1930, V. 4, No. 11, p. 422 and 427. A simple nomographic chart, one of a series for use in reinforced concrete design, for investigating diagonal tension in beams. Left hand scale, diagonal tension T; center scale, difference in moment between sections A and B; and right hand scale, depth d for various angles of stirrup. All in metric units, with explanation of use on chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating simple rectangular slabs and beams. JORGE QUIJANO. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 390-391.—A simple alignment chart, one of a series for use in reinforced concrete design. Left hand scale, depth of slab d, middle scales, moment M and area of steel A^s, and right hand scale width b, all in metric units. Explanation of use is given on the chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating slabs freely supported or partially restrained. JORGE QUIJANO. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 378-379.—A simple nomographic chart, one of a series, for use in reinforced concrete design to determine thickness of slab and area of steel. Left hand scale, span; center scale, depth of slab and area of steel; and right hand scale, load; all in metric units. This chart is for f_s = 20,000 lb. per sq. in., f_c = 900 lb. per sq. in. and n = 15. Explanation of use is given on chart and opposite page.—C. G. CLAIR AND M. N. CLAIR

Reinforced concrete bunkers in Germany. *Concrete Constr. Eng.* (England), Dec., 1930, V. 25, No. 12, p. 669-673.—During recent years large number of coal and coke bunkers have been constructed in Germany, some notable for long spans and heavy loads on main girders, and provision made

for possible settlement of ground due to mining operations. Total height of reinforced concrete bunker at Alsdorf is 170 ft. The capacity is 3,000 tons of coal. It is 78 ft. high and 79 ft. by 49 ft. 3 in. in plan. Due to the large space required by the cable drums, it was necessary to cantilever upper part of the structure on two sides. Foundation slab is 62 ft. 3 in. by 49 ft. 3 in. by 4 ft. thick with ribs projecting to a height of 11 ft. 6 in., thus reducing the maximum pressure on the ground due to dead and live loads and wind, to 3¾ tons per sq. ft. Owing to possibility of subsidence due to mining operations, the foundation was designed to resist stresses arising from uneven settlement of ground, and it is possible for whole structure to be inclined at angle of 5 degrees with vertical before dangerous stresses occur. Bunker is supported on four main columns, 16 ft. 5 in. by 4 ft. 1 in. cross section. Provision has been made for restoring structure to its original position if any settlement or canting takes place. Structural details of bunker are given.—JOSEPH MARIN

Angular rotation method for the analysis of multiple span frames. R. A. FREEMAN AND A. S. GILMAN. *J. Boston Society C. E.*, Dec., 1930, p. 543-562.—There is given the development of the fundamental equations for analysis of statically indeterminate frames based on the work of Ernest Suter entitled, "Die Methode der Festpunkte," published in 1923. Two numerical examples are worked out to illustrate the method. The angular rotation method is particularly valuable for analysis of multiple-span rigid frames.—MILES N. CLAIR

Thin reinforced concrete slabs. J. H. H. WILKES. *Roads and Rd. Constr.* (England), Dec., 1930, V. 8, No. 96, p. 434.—In case where steel is cheap, it may be more economical to employ floor slabs in bridges, or similar structures, in which the steel is greater than normal. In this case strength of slab will be determined by that of concrete. Curves show relationship between moment of resistance and percentage of steel, for various stresses in the concrete or the steel. Shear will be limiting factor for thickness.—JOHN E. ADAMS

Field inspection of concrete pipe culverts. R. W. CRUM. Iowa State College *Eng. Expt. Station* Bull. No. 99, Sept. 3, 1930.—In 1915, a question having arisen as to suitability of concrete pipe for use as highway culverts, Iowa Highway Commission and Engineering Experiment Station of Iowa State College cooperated in making a field study of such structures. Forty-six culverts ranging in size from 24 to 66 in. diameter were inspected. Inspections showed older designs to be inadequate, and since 1915 stronger and heavier pipe have been used. Records are given of certain culverts covering a life of 22 years, as oldest pipe examined in 1915 was then about 9 years old. Method of inspection was to crawl through pipe, making a thorough examination of all significant features by means of a flashlight. Principal opinions are: Concrete pipe made of good concrete and used in locations in which load on pipe does not exceed its supporting strength, and so installed that water flowing through (hydraulic traffic) cannot undermine support for the pipe, will make satisfactory culverts that can be classed as permanent as other concrete construction. The culverts examined did not exhibit progressive deterioration to an extent great enough, or in such a way, that predictions as to the length of life could be made therefrom. Culverts were found to be retired from service for two reasons—cracking, due to overload, and failure of lower end, due to undercutting by stream. Latter failure can be prevented by proper design and installation. Results of detailed examination of individual culverts are given. During the life of culverts covered by these inspections, pipe were installed without much reference to relation between strength of pipe and height of fill for particular locations, but investigation in 1915 showed that older pipe were not strong enough and at various later periods stronger designs were used. Necessity for standardization of materials and manufacturing

conditions as well as of design, is indicated by the fact that two plants working under the same design produced pipe of considerably different strength. Greatest destructive force acting upon concrete pipe culverts was pressure of fill supported by pipe including effects of truck wheels and other super-loads. Three defects in installation which affect the serviceability and durability of the culverts were: (1) open joints between sections; (2) culverts laid on too flat a grade line so that pipes became more or less filled with mud; and (3) culverts installed with outlets subject to undermining by cutting back of stream channel. The latter two defects might occur with any type of culvert. A point not to be overlooked is that although certain culverts may have various defects, they may still be performing their major function. Marston's method of computing probable imposed loads on culverts is included, together with curves for facilitating such computations.—R. B. B. MOORMAN

Foundation problems and their solution. L. R. VITREBO. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 301-303.—A footing is a structural unit used to distribute wall or column loads to foundation materials. A spread footing distributes load directly to soil without use of any other intermediate structural members. Such footings are nearly always made of reinforced concrete to save in excavating, materials and weight of footing proper. Five classes of spread footings are: (1) wall footings, (2) independent column footings, (3) combined footings carrying more than one column, (4) cantilever footings and (5) raft or mat foundation. In ordinary type of building, that footing should be chosen in which live load bears highest percentage to dead load, and its area should be determined for total load at allowable soil pressure. Pressure of dead load per unit of area should be determined, and area of all other footings should be proportioned for dead load only, with this unit pressure. Independent spread footings are usually square or rectangular for practical reasons. Those with sloping tops contain less concrete than stepped footings, but. unless slope is flatter than 2 horizontal to 1 vertical, the top will require forms which offset any saving in concrete. Combined footings are simply inverted girders, or slabs receiving an upward pressure equal to carrying capacity of soil, throwing their end reactions to the columns which bear upon them. Cantilever footings are often resorted to when combined footings are expensive or impossible to use, that is, when it is necessary to make use of land close to property line and still not encroach upon adjoining property. Raft or mat foundations are used for soils of low bearing capacity. The foundation is then spread over whole area of building using either flat slab or beam and slab construction.—D. E. LARSON

Reinforced concrete and its hypotheses. FERNAND DUMAS. *Génie Civil*, Dec. 6 and 13, 1930, V. 97, No. 23-24, p. 561-564 and 589-593.—Reinforced concrete theory is based on a series of paradoxical results of inexact and incomplete experiments. Various official circulars make no effort to conceal the crude approximations. Research has concentrated on elaboration of detail. Yet how much more important for the design of a section is exact determination of coefficient $n = 10$ or 15, than slight variations in allowable stresses? Design, instead of seeking to establish actual stresses produced by forces acting on a certain member, attempts only "to prove sufficient safety of structure," according to the German authority, Mörsch. The weakness of this procedure is admitted in 1906 specifications, requiring that tensile strength of concrete be taken into account under certain conditions. But what to do in the case of hyperstatic structures? A study of deformation of reinforced concrete structural members is greatly neglected. Deformation measurements with Manet-Rabut apparatus are accurate only in rare cases of simple tension or simple compression. Deflection measurements of flexural members yield results widely diverging from computed design values. The author attempts to throw light on the actual agreement of fundamental hypotheses of design with the basic properties of reinforced concrete and to test application of

elastic theory and theory of strength of materials in this case. Reinforced concrete is a heterogeneous material in its constitution and its fabrication. Its three constituents are: steel, concrete in tension and concrete in compression. Of these steel is least variable and should be—yet is not—standard of comparison. Early experiments by Considére and Bach show that properties of concrete are greatly altered in presence of reinforcement. Is it correct to apply theory of elasticity to this heterogeneous combination? Does law of continuity hold in this case? It seems, that elastic theory furnishes us only with approximate indications, which are to be treated only as average values. Heterogeneity due to fabrication upsets elemental theory of strength of materials, whose laws deal with simple tension, compression, flexure and torsion, on condition that shear is equal to zero. The assumption is that *some transverse section remains plane*, identical with its original position and perpendicular to neutral axis during deformation. The author takes up in succession compression, tension, flexure, transverse deformation and torsion of reinforced concrete and comes to the conclusion that the above assumption is inexact for reinforced concrete and that results of this hypothesis necessarily disagree with actual conditions.—M. A. CORBIN

Formulae and curves for the calculation of members in reinforced concrete in bending. F. CAMPUS.—*Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The formulae proposed permit making all bending calculations for ordinary members in reinforced concrete of rectangular or T section singly or doubly reinforced bending being eventually accompanied by compression or tension. In all these cases calculations are carried out by analogous formulae and the same coefficients of which values are given by five graphs. The simple manner in which the coefficients occur in the formulae shows clearly influence of various factors. These formulae can receive terms of adjustment to allow for compression in the ribs of T sections, and equally can be simplified to allow of rapid calculation. The formulae and curves are suitable for all values of coefficient of resistance and of elasticity of concrete and steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Calculation of rigid structures by the method of assumed static moments. J. RIEGER. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Method presented is called method of assumed static moments which consists of a geometric interpretation of equations provided by the theorem of Castigliano. A construction of several spans being given, it is always possible to divide it by the spans into several intermediate constructions for which conditions of deformation can be written as functions of assumed static moments with moment diagrams in relation to certain axes. These equations combined with static equations given the solution of the problem which is expressed in the general formulae by the form $M_m = \alpha_m A\ \beta_m B + \gamma_m C + \ldots \chi_m U + \varphi_m V + \xi_m Z$ in which the symbols indicate: M_m, moment in some joint of the construction given; $A, B, C, \ldots U, Y, Z$, certain functions of loads applied to the construction. $\alpha_m \beta_m \gamma_m \ldots \chi_m \varphi_m \xi_m$ being functions of the form of the construction studied. It is always easy to tabulate the functions $A, B, C, \ldots U, Y, Z$, and the coefficients $\alpha_m \beta_m \gamma_m \ldots \chi_m \varphi_m \xi_m$ to facilitate the use of the method.—EDITION LA TECHNIQUE DES TRAVAUX

Reinforcement of panels and slabs. H. LEITZ. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The progress of the use of panels and slabs as elements carrying load in reinforced concrete structures necessitates extension of theory of reinforced concrete to calculation of reinforced walls subjected to forces acting in their own plane, combined with the study of reinforcement of slabs. Under heading 1 (uncracked concrete) the usual hypotheses can be used to calculate the tensions

in the concrete and steel as well as for the arrangement of the reinforcement. The case of pure shear always occasions difficulty notably when reinforcements have finite thickness. The calculations for the reinforcement of a wall must be based on the hypothesis of cracked concrete (heading 2). The simplest and most certain reinforcement comprises two families of reinforcements following the lines of equal stress which at once raises difficulties on account of the curvature of these lines. For this reason two parallel sheets of straight reinforcement are very often used, and applied in various ways. The principal difficulties arise with the shear stresses. We have to admit that the concrete may act as a compressed diagonal in a direction inclined at 45 degrees to the reinforcement but we must expect large deformations following lines of principal tension. In case of heavy shearing loads and variable applications, three layers of reinforcement are necessary. Just as a deflected sheet may be considered as a pile of thin sheets, the same consideration may be applied to the corners of slabs simply supported.—EDITIONS LA TECHNIQUE DES TRAVAUX

ROADS AND PAVEMENTS

Concrete pavements. D. J. GARLAND, *J. Inst. Eng.* (Australia), Oct., 1930, V. 2, No. 10, p. 396.—The various well known methods of estimating proper thickness of concrete roads are reviewed, including Goldbeck's simple formula $h = \sqrt{\frac{3P}{f}}$ where h = depth of slab in inches, P = load in lb. and f = safe tensile strength of concrete in lb. per sq. in., also Prof. Westergaard's more elaborate treatment. In actual practice, a standard thickness is usually decided upon and foundations prepared where necessary. Since engineers have better knowledge of thickness of macadam required for various conditions, experimental results of comparisons between the thickness of macadam required and of an equivalent concrete slab are discussed. The results state that (a) for a given load intensity of pressure under a macadam road is 8 times pressure under concrete slab of same thickness, and (b) area of distribution of pressure under a concrete road is at least four times as great as under a macadam road of same thickness. These areas are actual areas over which pressure is recorded and not equivalent or effective areas. Result (a) appears more reliable. Accepting this data and assuming conical distribution of pressure to the foundations it is shown that if h' = macadam thickness and h'' = thickness of equivalent concrete slab, $h' = \sqrt{8h''}$. For greater safety the factor may be taken as less, say, 4, when $h' = 2h''$. In an example it is proposed to substitute a 5-in. concrete slab and prepared sub-grade for a 12-in. macadam surface. If $h' = 2h''$ equi valent macadam thickness for concrete slab = 2x5 = 10 in. and thickness of sub-grade required = 12 — 10 = 2 in.—JOHN E. ADAMS

ARCHITECTURAL DESIGN

The architecture of concrete. BERESFORD PITE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—Consideration of effect of primary distinction in method between an architecture of stone or brickwork and an architecture of concrete is motive of this paper. The idea of internal adhesion belonging to material producing homogeneity without anxious care must be novel, and of nature of a relief to the constructor; its attainment has always been his ideal, but the repugnance to the employment of concealed tierods for cohesion in walling is always manifested by the consistent tendency to rely either on trabeated or arcuated lines. The new idea, as our brethren on the Continent have perceived, is revolutionary rather than evolutionary. It raises manifold questionings as to

due architectural expression, and necessarily also it involves reconsideration of methods in study of practice of design, with courses of training leading to its fulfillment. It would appear to be consequential, if we had not experience of antiquity to the contrary, that a fundamental constructive method must deflect architectural expression of construction from traditional motives. Such expression is at least one of the primary causes of any architecture, though its caricature as decoration may survive and invade other materials and methods. Today we are in the presence of a new process that is rapidly becoming general and therefore little apology is required for an attempt to discuss its manifestation in architecture and its effect upon our ideals. The subject is sketched under three heads: 1. The difference in constructive method and its expression in design. 2. The resulting architectural forms and the possibility and service of ornament. 3. The problem of the aesthetic use of concrete architecture.—EDITIONS LA TECHNIQUE DES TRAVAUX

Architecture and reinforced concrete. E. BALIS. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—So weak is our poor spirit—said Suger—that it attains to truth only through the medium of realities perceptible to the senses. Truth is the spirit and the chief element of architecture, realities being the forms of the art. We may distinguish true and false forms. An unfortunate tendency of our times is to consider details while slighting the generality. This is a point brought out indicating our conception of new architecture and the forms proper of reinforced concrete. Architecture is an art and therefore something more than a technique.—EDITIONS LA TECHNIQUE DES TRAVAUX

The architecture of concrete and reinforced concrete. A. POMPE. *Summary Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930.—The aesthetics—still undetermined—of concrete and reinforced concrete are subject, like those of wood, brick and stone, to certain laws. These laws are themselves subject to the spirit of the time; they evolve in accordance with those material and immaterial conditions that govern the "directive" periods which follow on one another through history. That is to say, a "style" is far more an emanation from the spiritual and sentimental character peculiar to a given civilization, than it is the necessary and inescapable expression of the proper handling of a given material, the use of which happens to have been preponderant during that civilization. Style is an intermittent and periodic phenomenon, whereas the art of building is a permanent fact, though its value may vary. Style is expression: architecture is but a mask applied to building with more or less of expressiveness and truth. Wood, brick, stone, steel and reinforced concrete, each has its proper technique; that of rolled steel is a modern thing, and that of reinforced concrete is more recent still. In architecture steel has never properly succeeded, for a number of reasons. Reinforced concrete has only now entered upon a really active phase, and there is reason to believe, for other reasons, that this material will have an influence over the architecture of tomorrow much greater than that of its elder brother steel.—EDITIONS LA TECHNIQUE DES TRAVAUX

Four modes of construction in a single settlement or subdivision. *Betonwerk* (Germany), Dec. 7, 1930, V. 18, No. 49, p. 688.—Toward the end of year 1928 the Leipzig Fair Association presented a plan to the Sachsischen Arbeits and Wohlfahrtsministrium, to build four apartment buildings of 24 families each of steel, concrete, wood and brick respectively. The idea is to afford an opportunity to study each type of construction in a comparable manner. It is hoped that economical and technical phases of each class of construction may by this method be better observed. Each building is about 200 ft. long by 21 ft. wide and 3 stories high. The whole idea presents a building material exhibit in which many of building specialties can be seen. Each building is illustrated and described in detail. The entire enterprise is nearing

completion and is under direction of Rudolph Stegeman, Government Building Advisor, Leipsig. It is an experiment which up to this time has not been tried, and in due time the Leipsig Builders Fair will issue a complete report.—H. FRAUENFELDER

FIELD CONSTRUCTION

BRIDGES

Precast light weight concrete slabs solve bridge floor problem. *Concrete*, Jan., 1930, V. 37, No. 6, p. 15-16.—New type of bridge floor, consisting of large precast slabs of light-weight concrete welded to steel floor system, has been developed by Division of Bridges of City of Chicago. The Wabash Ave. bridge has slabs over the entire moveable part of the bridge floor, including roadway. Twenty-eight of largest size slabs, 102 sq. ft. in area, were required, twenty-eight 73 sq. ft. in area, and 56 smaller ones between street car tracks and rails. Thirty sidewalk slabs 98 sq. ft. in area and 2½ in. thick, were required. Roadway slabs have total thickness of 5¾ in. and are reinforced with steel bars in top and bottom, and stirrups. Sidewalk slabs, of light-weight concrete, are 2½ in. thick, and reinforced with layers of welded wire fabric, held 1 in. apart. Cement and aggregates employed were 1:1½:2½ mixture of Incor cement, No. 3 torpedo sand, and coarse haydite, with 4½ gal. water per sack of cement. Slabs were cast by aid of electric vibrating process that vibrated steel reinforcement. Molds were in horizontal position. Reinforcement was anchored firmly. Four holes were provided in each slab into which 1-in. pipe sleeves were fitted to facilitate handling, and later filled with mortar. Each slab was also anchored by U-bolts passing under stringers and up through holes in slabs where loose plates were held with nuts. Anchoring device removed after slabs were welded into place. Slabs were lifted into position by steam derrick equipped with two sets of cable spreaders fitted with hooks on each end. Final operation was welding of slabs to steel floor system, by means of ⅜-in. protruding steel plates.—C. BACHMANN

Winter concreting on a three arch bridge. RICHARD F. REY. *Eng. News Record*, Dec. 25, 1930, V. 105, No. 26, p. 1004-1005.—Continuous prosecution of concrete work on main-arch spans in severe winter weather and a nice procedure in handling expansion joints were employed in construction of Logan St. bridge in Lansing, Mich. The Logan St. Viaduct is 1,075 ft. long and has three river spans 61, 67, and 90 ft. long, 534-ft. of reinforced concrete viaduct, and 126-ft. of concrete-incased steel beam viaduct and earthfill approaches between retaining walls. Portions of bridge constructed in winter were housed in a framework covered outside with ship-lap siding sealed inside with building paper and roofed with tar paper. The housing provided ample space for building forms and handling concrete. Unit steam heaters kept enclosure warm. These were standard heater units such as are used for heating industrial and commercial buildings, instead of pipe coils or cast iron radiators. Outside temperatures varied from around freezing to 10° below zero, and inside temperatures were kept between 60 and 70° for first part of heating period and from 50 to 60° for last part. Concrete was made from aggregates heated by steam-pipe grids underneath stock piles and water heated by passing through old steam boiler. Temperature of concrete entering forms was between 60 and 80°. Vertical 1-in. expansion joints were carried across bridge over river piers and at 100-ft. intervals throughout viaduct portions. Butt joints were employed. Double columns and column girders were constructed throughout to eliminate any sliding expansion joints.—D. E. LARSON

BUILDINGS

The demolition of several reinforced concrete buildings at Brunn. R. BORTSCH. *Summary Reports*, First International Congress for Concrete

and Reinforced Concrete, Liege, 1930.—The machine factory of Brunn formerly Brand and Lhuillier disused for several years, was bought by the municipality which undertook in 1927 demolition of numerous buildings. By reason of existence of dwelling houses in immediate neighborhood, destruction of several halls and storied buildings in reinforced concrete of comparatively recent construction had to be undertaken with great care. The slabs were destroyed with a hammer. Ends of girders were cut with a pneumatic drill and reinforcement divided by an oxy-hydrogen flame. Frames of halls 13 meters high were overthrown in turn and cut up on the ground. Bases of columns were cut, then frames were overturned by hauling on cables with a windlass. Division of columns and girders of large dimensions was carried out after overturning by means of dynamite. For reasons of safety pieces to be destroyed were covered with hurdles held between metallic trellis work. Certain parts of buildings with high stories of 55 ft. were destroyed in one piece by dynamiting columns. Three months were taken in demolishing reinforced concrete buildings containing 12,000 cu. yds. Cost was 127,000 Belgian francs.—EDITIONS LA TECHNIQUE DES TRAVAUX

MISCELLANEOUS

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Jan., 1931, V. 38, No. 1, p. 35-40.—Second half of article on estimating labor and material quantities gives figures for definite areas for all units of form work. Hours of labor required are expressed as equivalent number of hours for one man. Lumber, other materials, and labor are each detailed. Time when each type of form may be removed is given.—C. BACHMANN

Concrete calendar. "Beton Kalender 1931." 1930, Wilhelm Ernst and Son, Berlin, 2 parts, 25th year, 1020 illustrations, R. M. 7.50.—Reviewed in *Bautechnik* (Germany), Sept. 19, 1930, V. 8, No. 41, p. 629.—This handbook for concrete and reinforced concrete construction is one of most common books and reference work for German contractors and concrete investigators. Two volumes are greatly enlarged and revised, especially chapters on slabs, tanks, arch bridges and girder bridges. Chapters on calculation of reinforced concrete cross-sections, solid floor construction and ground construction are new. New specifications for cement, mortars and concrete of several countries are given and great number of general tables are included. A. E. BEITLICH

New improved movable conveyors for cast concrete construction. RIEDIG. *Bautechnik* (Germany), Sept. 5, 1930, V. 8, No. 38, p. 567-8.—Great economy of movable conveyors for structural materials and their great field of application led more and more to their introduction in modern cast concrete construction. Two new patented conveyors are described with 66 and 44 tons per hour capacity. Materials are delivered to conveyor belt in first case by means of two screws and in the other case by means of a scraper. For concreting of quays, long walls or canals a new traveling concrete distributor is described handling 1400 cu. ft. per hour. A 2-yd. concrete mixer is mounted on base of crane. A number of minor improvements on travelers and chutes are illustrated.—A. E. BEITLICH

Crushed-stone plant builds profits by producing pre-mixed concrete. *Pit and Quarry*, Nov. 19, 1930, V. 21, No. 4, p. 56-58.—A new central-mixing plant has been erected in connection with crushing and screening portion of Hawthorne plant of Samuel Bracus Sons, Paterson, N. J. Eighteen reinforced-concrete storage bins of combined capacity of 5,000 tons and stock-piles assure a ready supply of aggregates. Two 3-compartment batcher bins are arranged side-by-side. Batching hopper of one bin can discharge dry mixes into transit-mix trucks or into a 3-cu. yd. stationary mixer, as desired. Mixer discharges

wet mixes into agitator trucks at one side. Other bin is for dry-batching stone, sand and cement, and it discharges into multi-batch trucks to be used in road construction. All proportioning is by weight. Sacked cement is received in truckloads and stored. Sacks are opened by hand and cement dumped through a grizzly in floor to conveyor which runs to a cement hopper at side of one batcher bin whence screw conveyor transfers the cement to weighing hopper. Firm operates ten dump-body and four transit-mix trucks.—A. J. HOSKIN

A caisson foundation job. O. E. MOGENSEN. *Civil Engineering*, Jan., 1931, V. 1, No. 4, p. 311-314.—The open-well type of caisson foundation, used extensively in the vicinity of St. Louis, is well illustrated by the foundations for the Missouri Portland Cement Company's new early strength Velo plant, built during 1927 and 1928. To ascertain practicability of this open-well method, it was decided to sink a concrete caisson to support the high concrete chimney, lined with fire brick at top. The chimney is 19 ft. in diameter, 220 ft. high, and weighs about 1,100 tons. The caisson which was 19 ft. 6 in. outside diameter, with a wall thickness of 3 ft., struck fire clay 42 ft. below the ground surface. The first section of caisson, cutting edge, was 10 ft. high, and was made from company's high early strength cement. The entire caisson was sunk quickly and without trouble. Other structures for which caisson foundations were used consisted of eight cement silos in clusters with interspace bins, each of the silos 32 ft. in diameter by 70 ft. high; four rock silos 35 ft. in diameter by 60 ft. high; and four clinker silos also 35 ft. in diameter by 60 ft. high. Materials to be stored in these silos weigh from 100 to 115 lb. per cu. ft. The three silo structures weigh, respectively, when filled, 32,620 tons, 16,640 tons, and 18,560 tons.—D. E. LARSON

Placing concrete under water. CAYETANO LUGO. *Ingenieria* (Mexico), Oct., 1930, V. 4, No. 10, p. 407.—Author contends that usual conception of action of the tube (tremie) in placing concrete under water is erroneous. He suggests careful attention to use of graded aggregates and of a number of tubes rather than one.—C. G. CLAIR AND M. N. CLAIR

ROADS AND PAVEMENTS

New system of concrete paving. *Rds. and Rd. Constr.* (England), Jan. 1, 1931, V. 9, No. 97, p. 24.—A system of paving is described, in which large, unsightly cracks are replaced by invisible hair cracks in much greater numbers. Instead of every precaution being taken to provide easy sliding of the slab over the foundations as expansion or contraction takes place, an unreinforced wearing surface is suitably anchored to a reinforced under-slab, which is again anchored into the sub-soil. In case of a clay sub-soil, for example, foundation is dusted with fine ashes or sand, and gridded with trenches 4 to 6 in. deep at say, 7 ft. 6 in. centers. A slab of 3 in. or more, thickness is poured and reinforced with at least 0.5 per cent steel. The top surface of this is indented with specially shaped elliptical recesses, and is provided with vertical pins for anchoring the top layer of concrete, and gaging its thickness. These pins also are used for indicating position of indent forms, and for holding reinforcement clear of ground. Top slab is then poured on this sub-base, and through indents and anchors, is made to break up into small squares each separated from one another by minute hair cracks caused by excessive tension. The ribs in the foundation cause a wide distribution of pressures, and continuity is provided. Less care need be taken in curing with this method of construction.—JOHN E. ADAMS

SHOP MANUFACTURE

Manufacturers like winter work. *Concrete Products*, Dec., 1930, V. 39, No. 6, p. 13.—A symposium of data from a large number of subscribers re-

garding the winter manufacture of concrete products shows that many manufacturers favor winter manufacture, at least on a reduced scale, for the purpose of holding their best men. It also allows them to accumulate stock, especially in case of block manufacturers, for an early spring building campaign. It is generally believed that the additional cost of products made in winter is negligible, although there may be some delay in the handling of frozen materials. The opinion is expressed by one burial vault manufacturer that winter is a better season for the burial vault business, and it is his belief that vaults cured in winter are of better quality.—E. S. HANSON

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, Jan., 1931, V. 38, No. 1, p. 17-20.—An illustration of the production ills of a large manufacturing concern are described and the prescribed remedial measures outlined. Interest to products industry in the example is possibility of reducing stocks carried, by smoothing out production curve with fluctuating delivery curve, by a careful control of a few major items of plant's product. As an application of method to the products business, the output for one year, based on the actual experience of a plant, is given in the first four columns of a table reproduced. The second column shows the number of the item in column (1) sold during the year. The third column converts amounts in column two to a common base. Fourth column shows production in "equivalent units" for each item, expressed as percentage of total production, a glance showing that the major production was in five items out of sixteen listed. By including in Group A all items over 5 per cent of total production, and in Group B all items under 5 per cent, we find 90.28 per cent of the production in five items of Group A and 9.79 per cent in 11 items of Group B. By study of production and stock, so far as five items of Group A are concerned, control of the major part of the output is accomplished and items in Group B can be handled by simple method as to stock carried and as a fill-in at convenient times, as to production. Minimum stock is determined as follows: First, the total stock carried shall be equal to the maximum month's sales for preceding year, or estimated maximum month's sales for coming year, if they are expected to vary materially from preceding year, to which is added one-half month's production. Second, stocks for each of major items, Group A, should be equal or nearly equal to maximum month's sales of that item. The stock for the items in Group B should be carried on average about one-half of an economical production quantity.—C. BACHMANN

Hollowspun concrete piles in the making. H. F. HEDDERICH. *Concrete Products*, Nov., 1930, V. 39, No. 5, p. 12-14.—Method followed by Westinghouse Electric & Mfg. Co., in making concrete piles is given in detail. Company has for past six years been producing concrete lighting and trolley posts by the centrifugal method and has recently applied same method to manufacture concrete piles. Piles are round and uniformly tapered for their entire length, increasing ¼ in. in diameter for each foot of length. The ultimate strength of the pile is governed by the compressive strength of the tip. The design is such that failure will occur at this point before the pile is stressed to its limit as a column. All of piles are provided with facilities for jetting. Concrete mix consists of one part cement to three parts of combined aggregate. Water is controlled to give concrete proper consistency for placing and spinning. Material used for both coarse and fine aggregate is very hard, highly dolomitic limestone with cubical fracture. It is very thoroughly washed during its preparation, and is received entirely free of dust. Mold is rotated at 400 r. p. m. for ten minutes. During the process of spinning, the solid particles (the aggregate and cement) are driven outward by centrifugal force and uniformly compacted against the inside surface of the mold and around the reinforcing. This brings all the entrapped air, a large amount of water, and some of the finer aggregate and cement, to center. A hole is formed throughout the length of the column by this action, and as the spinning continues most of the fine solids are packed around periphery of this hole.—E. S. HANSEN